WATER RESOURCES DEVELOPMENT PROJECT

PARK RIVER
LOCAL PROTECTION

CONNECTICUT RIVER BASIN
HARTFORD, CONNECTICUT

DESIGN MEMORANDUM NO. 9

AUXILIARY CONDUIT TUNNEL

SITE GEOLOGY, FOUNDATIONS, CONCRETE MATERIALS AND DETAILED DESIGN OF STRUCTURES

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS.

DECEMBER 1976

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM, MASSACHUSETTS 02154

REPLY TO
ATTENTION OF

NEDED-E

12 January 1977

SUBJECT: Park River Local Protection, Connecticut River Basin, Hartford, Connecticut, DM No. 9, Auxiliary Conduit Tunnel, Site Geology, Foundations, Concrete Materials and Detailed Design of Structures

HQDA (DAEN-CWE-B)
WASH DC 20314

1. In accordance with ER 1110-2-1150, there is submitted for review and approval Design Memorandum No. 9, Auxiliary Conduit Tunnel, Site Geology, Foundations, Concrete Materials and Detailed Design of Structures for the Park River Local Protection, Connecticut River Basin, Hartford, Connecticut.

2. The design included in the DM has been developed using the Draft of Guide Manual EM 1110-2-2901, Engineering and Design, Tunnels and Shafts in Rock, dated December 1973, with revised Chapters 1, 2 and 3, dated 5 November 1976.

3. It is requested that specific response be made to recommendations contained in Paragraph 21, Recommendations.

FOR THE DIVISION ENGINEER:

George T. Sarandis

Incl (10 cys)
as

GEORGE T. SARANDIS
Acting Chief, Engineering Division

WATER RESOURCES DEVELOPMENT PROJECT

PARK RIVER LOCAL PROTECTION
CONNECTICUT RIVER BASIN
HARTFORD, CONNECTICUT

DESIGN MEMORANDA INDEX

Number	Title	Anticipated Submission Date	Date Submitted	Date Approved
1	Hydrology		16 Feb 73	12 Apr 73
2	GDM - Phase I - Plan Formulation		30 Mar 73	16 Jul 73
2	GDM - Phase II - Project Design, Site Geology & Interior Drainage Part I - Box Conduit		30 Aug 74	18 Oct 74
2	GDM - Phase II - Project Design Part II - Auxiliary Conduit		24 Jan 75	3 Mar 75
3	Hydraulic Analysis		19 Feb 75	27 Mar 75
4	Concrete Materials Part I - Box Conduit		22 Apr 75	6 May 75
5	Embankment & Foundations Part I - Box Conduit		7 Feb 75	18 Mar 75
6	Pumping Stations		29 Nov 74	10 Jan 75
7	Detailed Design of Structures Part I - Box Conduit		25 Oct 74	4 Dec 74
8	Auxiliary Conduit Shafts Site Geology, Foundations & Detailed Design of Structures		30 Sep 76	15 Nov 76
9	Auxiliary Conduit Tunnel Site Geology, Foundations, Concrete Materials & Detailed Design of Structures		12 Jan 77	

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SITE GEOLOGY, FOUNDATIONS, CONCRETE MATERIALS AND DETAILED DESIGN OF STRUCTURES

CONTENTS

LIST OF PLATES

TABLES

LIST OF APPENDICES

WATER RESOURCES DEVELOPMENT PROJECT

PARK RIVER LOCAL PROTECTION
CONNECTICUT RIVER BASIN
HARTFORD, CONNECTICUT

A. PERTINENT DATA

PURPOSE	Flood Control
LOCATION	
State	Connecticut
County	Hartford
City	Hartford
River	Park River & North and South Branch Park River
River Basin	Connecticut
PARK RIVER DRAINAGE AREAS	
Park River at the Mouth	78.7 Square Miles
North Branch Park River	27.7 Square Miles
South Branch Park River	47.0 Square Miles

RECORD OF MAJOR FLOODS

Year	Month	Peak Discharge, c.f.s.*
1936	March	5,400
1938	January	5,650
1938	September	5,320
1955	August	14,000
1955	October	6,420

*Gage at Riverside St. on Park River about 600 feet below junction of North and South Branches.

AREAS

Subject to flooding, Acres	3,000
Inundated 1955 flood of record, Acres	2,300
Properties protected	Industrial, commercial, residential and public

TWIN-RECTANGULAR BOX CONDUIT

Material — Reinforced Concrete

Conduit Section	Length in Feet	Half Section Width	Half Section Height
Existing (12,743 ft):			
Original (1944)	5,600	30'-0"	19'-4"
Section 1	1,213	34'-0"	26'-6"
Section 3	1,710	34'-0"	26'-6"
Section 6	1,460	36'-0"	27'-6"
Section 8	2,760	22'-0"	25'-0"
UNDER CONSTRUCTION (4,036 ft): (Scheduled completion date - 1980)			
Section 2	1,232	34'-0"	26'-6"
Section 4	1,337	34'-0"	26'-6"
Section 5	103	36'-0"	27'-6"
Section 7	1,044	22'-0"	25'-0"
Section 9	320	22'-0"	25'-0"

POPE PARK PUMPING STATION - (Formerly Riverside Pumping Station)

Location	Pope Park by Conduit Section 4
Capacity, CFS	75
Area Controlled, Acres	40
Pumps (No.)	3

ARMORY PUMPING STATION

Location	State Armory by Conduit Section 1
Capacity, cfs	120
Pumps (No.)	3

AUXILIARY CONDUIT

Length, feet	9,100
Size, Inside Dia.	22'-0"
Liner Material	Reinforced Concrete
Capacity	5,400 cfs
Predominant Excavation	Shale Rock

SYSTEM DESIGN CAPACITY (Conn. River Stage of 30 ft. MSL)

Park River Conduit	18,400 CFS
Auxiliary Conduit	5,400 CFS
Total	23,800 CFS

AUXILIARY CONDUIT SHAFTS

Intake Shaft

Inside Diameter	22'-0"
Depth, Total, Feet	143
Depth to Rock, Feet	64
Liner, Material	Reinforced Concrete

Outlet Shaft

Inside Diameter in Soil	30'-0"
Inside Diameter in Rock	22'-0"
Depth, Total, Feet	146
Depth to Rock, Feet	70
Liner in Soil, Material	Reinforced Concrete
Liner in Rock, Material	Reinforced Concrete
Height of Protection During Construction	Elevation 27.0 MSL

Local Drainage Shafts

Inside Diameter	6'-0"
Liner Material	Reinforced Concrete
Total Depth, Broad St. Shaft	175'
Depth the Rock, Broad St. Shaft	50'
Total Depth, Main St. Shaft	185'
Depth to Rock, Main St. Shaft	20'

B. INTRODUCTION

1. PURPOSE. The purpose of this memorandum is to present for review and approval the geotechnical, structural and materials investigations and design criteria developed for the Auxiliary Conduit Tunnel of the Park River Local Protection Project, Hartford, Connecticut. Basic criteria, loading assumptions, typical computations, logs of explorations, laboratory data, definitive plans and other pertinent data are presented.

2. SCOPE. This memorandum incorporates the geotechnical, structural and concrete materials investigation aspects associated with the design and construction of the Auxiliary Conduit Tunnel. The shafts, intake and outlet structures appurtenant to the tunnel were presented in Design Memorandum No. 8 submitted September 1976. A description of the entire project is contained in Design Memorandum No. 2, Phase II, Project Design, Part II - Auxiliary Conduit. Scheme D of that memorandum, the deep rock tunnel, is now the adopted scheme and is the basis of this memorandum. Hydrology and hydraulics for the project were submitted in memoranda which have been approved and are referenced herein.

3. STATUS OF PROJECT. Part I of the project, consisting of an inlet structure at the North Branch of the Park River, five sections of twin box conduit, a junction structure at the convergence of the North and South Branches of the Park River and two pumping stations is now under construction. The construction of the Auxiliary Conduit, Part II of the project, is expected to commence in 1977 and continue for a period of three years.

4. DESCRIPTION OF TUNNEL. The Auxiliary Conduit Tunnel will conduct approximately one quarter of the maximum flow in the Park River from the convergence of the North and South Branches of the River (Junction Structure) to the Connecticut River at a point 1200 feet downstream from the outfall of the twin box conduit enclosing the Park River. The completed tunnel will have a 22 foot inside diameter and extend some 9100 feet between the intake and outlet shafts. It will be excavated through shale rock and basalt at a maximum depth of 200 feet below ground surface. The tunnel invert at the outlet shaft is 52 feet below the intake invert with the tunnel sloping at a rate of approximately seven inches per 100 feet. A minimum rock thickness of approximately 50 feet will remain above the crown excavation at the outlet. The tunnel will be lined throughout the reinforced concrete. Plate 9A-1 is a general plan of the project and Plates 9A-2 through

9A-19 are detailed plans and profiles of the proposed tunnel. Plate 9A-20 contains moment and thrust diagrams for the tunnel liner and Plates 9A-21 and 9A-22 show details of the rock support systems and tunnel liner.

5. DEVIATIONS FROM PREVIOUS MEMORANDA. The Auxiliary Conduit Tunnel as presented in this memorandum differs in several respects from that presented in Design Memorandum No. 2, Phase II, Part II. As noted in Design Memorandum No. 8 the cleanout and inspection shaft has been eliminated and two small diameter local drainage shafts have been added. The transition from square to circular cross-section is accomplished in the intake shaft making the tunnel circular in section throughout. The minimum design thickness of cast-in-place liner has been reduced from 24" to 13" as a result of deepening of the tunnel and more complete geologic data.

6. CONSTRUCTION ALTERNATES. Three tunnel lining systems will be included in the contract documents with bidding schedules for each. The alternates will be as follows:

1. Cast-in-place liner with drill and blast excavation.
2. Cast-in-place liner with mole excavation.
3. Precast segmental liner with mole excavation.

The least expensive of the alternates will be selected.

7. DISPOSAL OF EXCAVATED ROCK. The majority of rock excavated for the tunnel will be transported to the primary spoil area located north of the intake shaft as shown on Plate 9A-1. The area is low lying and contains approximately 13.5 acres. The excavated material will raise the elevation of the area so that it will not be subject to ponding thereby enhancing prospects for future development. A second spoil area will be designated at the Department of Public Works yard in Hartford for the excess material not required at the primary area.

8. TEMPORARY ROCK SUPPORT. The cast-in-place liner alternatives will require rock support for the entire length of the tunnel. Rock bolts with shotcrete will be employed at varying spacing, depending on the quality of rock encountered during construction, for all but the reach of tunnel within the fault zones in the vicinity of stations 57+50 and 95+0. Specified minimum and anticipated maximum rock bolt patterns for the tunnel crown have been shown on Plate 9A-21. The reaches within the fault zones, approximately 300 hundred feet in length, will be supported by W 8 steel ring beams at an anticipated average spacing of

three feet. The actual spacing will be determined during construction. Shotcrete coatings will be specified with each of the above rock support systems. Rock bolts at close spacings will be used in lieu of ring beams in the cast-in-place liner with mole excavation alternate. Temporary rock support will not be required if the precast liner alternative is selected. The term "rock bolt" for purposes of this memorandum refers to the rock reinforcement described herein although under more strict definition they should be described as rock anchors.

C. GEOTECHNICAL DESIGN

9. PHYSIOGRAPHY. The Park River at Hartford, Connecticut, is in the Connecticut Valley lowland, a well-defined depression within the New England Upland subdivision of the New England province. The elongated valley trends roughly north-south and is drained by the Connecticut River. Hartford is in the middle of the valley, which is about 20 miles wide at that point. The lowland is underlain by interbedded Triassic conglomerate, sandstone and shale with sheets of trap rock and is flanked by more resistant igneous and metamorphic rocks of the highlands. The trap rock is more resistant and stands out from the sedimentary rock types as ridges because of differential erosion. Various glacial deposits cover the area.

10. REGIONAL GEOLOGY. In this vicinity the valley is a structural feature, probably a graben with north-south trending major faults on the east and west margins. The Triassic sequence in the valley consists largely of thick interbedded red arkoses, shales and conglomerates which dip easterly at about 15° to 20°. Within the formation are beds of mostly red shales together with basalt flows and diabase dikes and sills. These types strike northerly and outcrop in a narrow belt running up the valley north of Hartford, in a zone about 10 miles wide just south of Hartford, and again in a narrow zone extending to the south. Numerous fault blocks are recognized mainly in and around the bodies of trap rock. The rocks are not known to be folded significantly. The region is blanketed by glacial till which occurs at the surface in the higher parts of the lowland. However, in the low, flat or gently undulating plains which form the surface throughout most of the region, the till is overlain by widespread glacial lake deposits of stratified sand and varved silt and clay. The lake beds are buried locally by glacial outwash deposits.

11. SEISMICITY. The project is located in zone 1 on the seismic risk map accepted by the Coastal Geoditic Survey. There may be possible minor damage to structures from earthquakes in zone 1 areas. In accordance with Engineering Technical letter No. 1110-2-109 dated 21 October 1970, hydraulic structures in zone 1 will be designed to withstand earthquake accelerations of .05 g. As faults in the area are not considered to be active, seismic forces will not be considered in design of the tunnel linings as discussed in chapter 3 of the revised draft of EM 1110-2-2901.

12. PREVIOUS TUNNELING IN THE AREA. Under the direction of the Bureau of Public Works for the city of Hartford a 6'0" wide by 6'2½" high horseshoe shaped tunnel called the "Jefferson Street Tunnel" was constructed in 1934 and 1935. This tunnel which was designed to be 7,554+ feet in length was constructed prinicpally in rock with short earth sections on both ends. The tunnel is located approximately 1000 feet south and in a generally parallel attitude to Park River Auxiliary Conduit. The tunnel has a design invert elevation varying between +24.27 and +12.66 at a grade of 0.13%. In comparison the invert grade of the Park River auxiliary is from -95.07 to -147.0 MSL, a depth of approximately 120 to 150 feet below the Jefferson Street Tunnel. Construction records of the tunnel are very sketchy; however, the only recorded major problems during construction were in areas of tunneling in clay where air was required to advance the tunnel and in areas where the tunnel broke through the rock surface and required unanticipated earth tunneling methods and associated support. The geologic conditions for design were evaluated by Charles P. Berkey for the Bureau of Public Works and are recorded in a report entitled "Report on the Geologic Features of the Jefferson Street Tunnel" dated March 24, 1934. Available data on the actual design and construction of the tunnel is very limited with information primarily related to claims in the overburden, alleged vibratory damage and inspector's field notes. No official as-built record of construction has been discovered. All available information in regard to this work has been considered in the design of the Park River Auxiliary Conduit.

13. SUBSURFACE INVESTIGATIONS.

a. Current Investigations. Explorations consisted of core borings, various tests within the boreholes and a seismic survey. Tests in boreholes included borehole photography, pressure testing, piezometers, observation wells, and pump tests.

(1) Core Drilling. Rock cores from 29 borings were used to determine tunnel geology; 18 were M-series chrome barrel NX-diameter cores and 11 were M-series 4-inch diameter. (See Plates 9A-2 thru 9A-4 for locations.) Ten boreholes do not reach tunnel grade because they were drilled for a previous design in which the tunnel was higher in elevation. Rock cores were logged, classified, tested, and photographed in the laboratory. All cores were photographed in the field immediately upon removal from the core barrel for future use during contract bidding. A typical drill log and photographed box of core are shown in Appendix A.

(2) Borehole Photography. At the time of this memorandum 4 boreholes reaching tunnel grade and 11 shallow boreholes (before deepening) have been photographed with the borehole camera to determine rock structure. The logs indicate joint orientations and depths of rock structure. Pertinent orientation and frequency rosettes and polar diagrams are plotted on Plates 9A-5 thru 9A-19. Joint analyses were prepared from these data, examples of which are in Appendix B. Borehole photography is continuing for complete coverage of borings extending to the depth of tunnel grade.

(3) Pressure Testing. Boreholes were pressure tested except those in which artesian flows occurred. The procedure consists of lowering a perforated pipe that normally sets off a 5-foot zone by air inflatable or mechanical packers. The packers are expanded at depth and then water flow into the borehole is measured at each successive zone. Pressure test results are shown graphically next to the borings on Plates 9A-5 thru 9A-19. Pressure testing is continuing on all borings not previously tested.

(4) Observation Wells. Six boreholes were left open and were used as observation wells to record ground-water levels. Results are tabulated in Appendix C. Artesian flows were measured in some boreholes during the explorations and are noted on detail Plates 9A-5 thru 9A-19.

(5) Pump Tests. Selected boreholes were pump tested, usually at tunnel grade, to determine water inflow. The summary of pump test data is shown in Appendix C.

(6) Piezometers. Readings from existing piezometers are shown in Appendix C. Other piezometers are to be installed in selected boreholes after forthcoming borehole photography is completed. Each installation will consist of several piezometers installed at various horizons at or near tunnel grade.

(7) Siesmic Survey. A seismic survey was conducted under contract along the tunnel alignment to determine depths to rock and to aid in selecting borehole locations. Seismic velocities and rock surfaces are incorporated in Plates 9A-5 thru 9A-19.

b. Future Investigations. Additonal drilling is planned around the site of the caisson at the outlet. Six wash borings are scheduled. In situ testing to determine stresses at tunnel grade is planned and it will be followed by a 100-foot test section at the start of the tunnel during construction. The in situ testing will be done cooperatively between NED, WES and the U.S. Department of Transportation.

14. <u>LABORATORY TESTING</u>

a. General. Rock core samples for testing were chosen primarily from zones within the tunnel, near the crown and within one-half diameter above the crown. Some samples from higher elevations were tested during the earlier design. Tests were made to determine factors such as densities, rock strengths, modulus of elasticity, Poisson's rations, failure characteristics and joint strengths. The test results give parameters that are applied to designs of supports and determinations of tunneling characteristics. As only competent samples can be tested some of the weakest and poorest quality zones may not have specific test values. The testing program is continuing and final design will be based on the completed testing program.

b. Tests. The tested zones are shown on Plates 9A-2 thru 9A-4 Appendix D contains summary descriptions of the tests and tables of results. In summary the types of tests conducted on rocks included density measurements in all zones, unconfined compression tests, unconfined compression tests with strain measurements, dynamic tests, triaxial shear tests, multi-stage triaxial shear tests, and sliding friction tests on natural bedding joints. The intent is to obtain representative values for the variety of rock types and conditions encountered. A summary of selected properties follows on Table 1. Significance of individual results are discussed for each rock type under "Physical Rock Properties" paragraph 16.

TABLE 1

SUMMARY OF MECHANICAL ROCK PROPERTIES

		RED SHALE	GRAY SHALE	BASALT	APHANITE	RED SANDSTONE
SPECIFIC GRAVITY (DRY)	TESTS COUNTED	25	4	14	3	2
	RANGE	2.58 - 2.72	2.61 - 2.73	2.68 - 2.87	2.46 -2.62	2.58-2.73
	AVE.	2.66	2.66	2.74	2.54	2.66
UNIT WEIGHT (PCF)	TESTS COUNTED	25	4	14	3	2
	RANGE	161 - 169.7	162.9 - 170.4	167.2 -175.3	153.5-163.5	161 -170.4
	AVE.	166	166	172.2	158.5	165.7
UNCONFINED COMPRESSIVE STRENGTH (PSI)	TESTS COUNTED	19	4	11	3	2
	RANGE	3,242-13,100	4,329-14,740	5,540-13,740	2,700-6,660	9,350-9,536
	AVE.	7,752	8,556	10,263	4,090	9,443
MODULUS OF ELASTICITY E(psi x 10^6)	TESTS COUNTED	7	1	9	1	
	RANGE	.2 - 5	2.5	.89 -10.	3	-
	AVE.	2.1		4.62		

NOTES:

1. Testing is continuing
2. See Appendix D for complete table

15. TUNNEL GEOLOGY.

a. Stratigraphy. The rocks along the alignment are primarily easterly dipping Triassic sandy shales/siltstones interrupted by a zone of basalt flows and some limited unique rock types near the basalt. The interpreted stratigraphy is shown on Plates 9A-2 thru 9A-4 at an exaggerated scale; details are shown on geologic log profiles, Plates 9A-5 thru 9A-19 which are at a natural scale. Bedding is normally distinct and often regular to the extent that many marker beds correlate between boreholes. Structural anomalies are present as shown on the plates. Descriptions of the various significant rock types are as follows:

(1) Red Shale/Siltstone. The dominant rock type is reddish-brown shale/siltstone. The shale contains sandy phases and is interbedded with gray shales and thin sandstones. It is thin bedded and calcareous. Calcite fills the open bedding planes, joints, and fractures. The shales are usually well-cemented and moderately hard but some zones are classified as soft and weak. The sandy phases are mostly competant and hard to very hard. Shale samples from near the intake exhibited a slaking-like action when submerged. This is attributed to stress relief by coring (see paragraph 16.g). Bedding strikes roughly north-south and generally dips 10° to 20° to the east but with local variations.

(2) Gray-Black Shales. Gray and sometimes black shales are interbedded with the red shales. They are thin-bedded and similarly oriented. The beds are thinner than the red beds and were used as markers to correlate between boreholes. Gray shales are calcareous, moderately hard to soft and are similar in physical properties to the red shales.

(3) Sandstones. Thin whitish to gray calcareous sandstone beds are common within the shales. Many sandy zones appear to correlate between boreholes and were used as markers. The beds are hard but sometimes show some solution activity and localized concentrated jointing. Variations include a coarse red sandstone (arkose) and a thin zone of interbedded volcanic sandstone and shale that were encountered in boreholes FD-9T and FD-24T respectively, but in no other borings.

(4) Basalts. Basalt flows near the intake shaft are oriented consistent with the local stratirgaphy although structural modifications are apparent (see Plate 9A-2). They are usually gray and olive gray (locally black), slightly vesicular and non-vesicular, calcareous, hard and contain healed hairline fractures throughout. Localized broken and weathered zones occur and are noted on the drawings.

(5) Aphanite. This gray fine-grained to glassy rock type occurs in borehole FD-9T between the depths 137 and 188 feet. Its origin is uncertain and it occurs in zone with unresolved structural discontinuities. It is hard to very hard but also contains numerous irregular healed hairline fractures. Some zones may be slightly weathered and less dense.

b. Structural Geology

(1) Jointing. Joints were examined for characterisitics in rock core and also logged and oriented by borehole photography. Detail sections on Plates 9A-5 thru 9A-19 show singificant joint data where available. Joint orientation and frequency data are still to be obtained from additional borehole photography at tunnel grade. The prinicple joint control applicable to each of the rock types is as follows:

Shales. The primary joints in the shales are along the bedding generally dipping between 10° and 20°. Steeply dipping joints are variably distributed as strike and dip joints with irregular to rough surfaces and dips ranging from 70° to 90°. Many are very thin and healed with calcite.

Sandstones. Specific joint sets were not identified in the sandstone beds and are considered for design purposes to be random and generally steeply dipping.

Basalt. In borings FD-8T and FD-14T less than 10 percent of the joints were parallel to the trend of bedding in the shale. More than 50 percent strike between N45°E to N45°W and dip 60° to 90°.

(2) Folds. Folded beds appear present at station 57+50. Borehole photography detected a reverse dip of the beds in FD-22T. See Plate 9A-11. The photography is incomplete as the borehole has been deepened since being photographed. A fault is considered likely in this zone based on the core samples. Its presence is to be verified by the completed borehole photography.

(3) Faults. Faults are interpreted in two areas, near Station 57+00 and between Stations 89+50 and 95+50. Other broken zones are as shown on the plates. At Station 57+00, the fault is based upon reversed direction of dip in the upper beds and an apparent shear zone, See Plate 9A-11. The fault is assumed to have a trend parallel to the basalt bodies. The extent of the fault zone has not been fixed and is inferred from this local geologic structure. Faulting in the zone between Stations 89+50 and 95+50 is suggested by the anomalous steep dips of upper beds and disrupted stratigraphy between basalt and shales. The structural interpretation is shown on Plate 9A-5 and is based on the rock cores. The actual structure is not known. Borehole photography is not considered practical as a tool to delineate further interpretation of the rock structure in this zone.

(4) Weathered Zones. Weathered zones are variable throughout the section. Significant areas of weathering are shown on detail Plates 9A-5 thru 9A-19 prepared from the rock core logs.

c. Special Geotechnical Considerations.

(1) Stations 23+00 to 31+00. Special geotechnical considerations are necessary between Stations 23+00 and 31+00 where the tunnel intersects an area of thin rock cover and thick overburden and conditions at tunnel grade are described as very blocky and seamy. (See Plates 9A-15 and 9A-16). Within this reach there are large scale on-lap/off-lap stratigraphic "wedges" consisting of highly crossbedded and intercalated shale and sandstone units (from laminae to thin beds). Although generally classified as shale or "shaley", the units tend to be more brittle because of abundant limey cement. Unit apparent dips in the tunnel section in this reach range from 7 degrees to 14 degrees with local preconsolidation crossbeds to 25 degrees. Unlike conformable shale units which could simply slide when stressed, the unconformable sandy "wedges" probably developed arching and tensioning which caused a very blocky and seamy condition in this reach, that is a nearly isotropic condition. There is also minor leaching and surficial staining to unknown depths, but no "gouging" or other evidence of faulting. Generally, for design purposes there is easily sufficient rock to rock wall contact with no detrimental minerals or clay to reduce friction. The rock, however, is not tight and did take water under applicable head pressures to a maximum of 15 gpm per 5-foot zone during pressure testing - even at depth. Analysis of the above conditions with detailed core logging in accordance with Barton, Lien and Lunde (Rock Mechanics 6, 189-236) utilizing rock block size, interblock shear strength, and active stress (a function of rock load and water inflow) predicts a normal rock pressure of 1.1 TSF for this reach as shown on Table 2 and supported by computations in Appendix E for a

condition described as worst average. This value is less than that devised by the modified Terzaghi approach presented in the Corps manual which would arrive at a value of 2.2 TSF. It is anticipated, therefore, that the only real problem in this reach may be surficial spalling so that some barring down and/or supplemental bolts may be necessary, especially where there are locally higher water inflows of short duration.

(2) Stations 93+50 to 95+50. Broken rock assumed to result from faulting is anticipated between Stations 93+50 and 95+50 (See Plate 9a-2). The mixed rock types are disrupted stratigraphically. Upper beds in FD-9T dip 60° and the lower beds dip 20°, consistent with the regional trend. RQD'S are about 30. Pressure test takes were 15-20 gpm. Table 2 shows that the load by the Barton, Lein and Lunde criteria is 1.85 TSF requiring bolts, wire mesh, and shotcrete for support. Criteria from EM-1110-2-2901 predict a load of 4.8 TSF requiring steel sets and shotcrete.

(3) Stations 56+00 to 57+00. A fault zone is projected into the tunnel from the broken zone logged in boring FD-22T at about Station 57+50 (See Plate 9A-11). Loading and support considerations would be similar to those for the zone between 93+50 and 95+50 discussed above.

16. PHYSICAL ROCK PROPERTIES.

a. General. A table of the physical properties of the rocks tested along with examples of the test results and procedures are in Appendix D. An example of a detailed log of the rock core classified as to rock structure in accordance with the report by Barton Lien and Lunde "Engineering Classifications of Rock Masses for the Design of Tunnel Support" is included in Appendix A. This classification has been used in computing an alternate series of tunnel loading factors and is shown in Table 2.

(1) Sample Selection. Samples were selected from 21 explorations on the tunnel alignment and were generally distributive of the area of tunnel section. The sample distribution of testing in relation to the tunnel profile is shown on Plates 9A-2 through 9A-4. Testing is not completed and a continued updating of the design will be made during plans and specifications as test data becomes available.

RECOMMENDED ROCK LOAD CONFIGURATION
PARK RIVER AUXILIARY CONDUIT

Rock Quality Designation	EM-1110-2-2901 (1973 Draft) Load T.S.F.	EM-1110-2-2901 (1973 Draft) Rec. Support	Barton, Lein & Lunde (1) Load T.S.F.	Barton, Lein & Lunde (1) Rec. Support	Machine Excavation EM1110-2-2901 (1973 Draft) Load T.S.F.	Machine Excavation EM1110-2-2901 (1973 Draft) Rec. Support w/o Pre-Cast	Depth Ft. Invert	Est. Tunnel Length	Water Inflow CPM/L.F.	Assumed Over-break	Rock Stand up Time
Best Avg. RQD=80+	1.1	(utg) 11'B 4.5' C.C. w/ 1.0" S	.52	(utg) 9'B 6'C.C. to Sb w/1.0" S	.5	(utg) 10'B occ. to 6.0' c.c. w/1" S as req.	50% 160 50% 180	8,000	1	8"	6 mos for 12' span
Worst Avg. RQD=40	2.2	(utg) 11'B 2.25' C.C. w/ 2.0" S	1.1	(utg) 9'B 3.0'C.C. w/10" S	1.4	(utg) 10'B 3.0'-5.0' C.C. w/2" S as Req	150	800	4	10"	1 wk for 9' span
Fault Zone RQD=30	4.8	SS 2.0'-4.0' C.C. w/ 3.0" S	1.85	(tg) 9'B 3.0'C.C. w/Mr 2"-5"S	3.5	(utg) 10'B 3.0'C.C. w/3.0" S	180	300	-50 est	12"	5 hrs for 4.5' span

(1) From Report entitled "Engineering Classification of Rock Masses for the Design of Tunnel Support".

Key to Recommended Temporary Support Systems

B = Systematic bolting
Sb = Spot bolting
(utg)= Untensioned, grouted

(tg) = tensioned and post-tensioned
S = Shotcrete
Mr = Mesh reinforced

CCA = Cast Concrete Arch
Sr = Steel reinforced
SS = Steel Sets

(2) Density. The density of the rock averages 2.66 BSSD (166 pcf) for the shale and 2.75 BSSD (171.6 pcf) for the basalt. These rocks comprise the bulk of the rock to be encountered during the tunneling and these values have been utilized for design loading. Two less prominent rock types shown on the stratigraphic profile have densities of 2.66 BSSD (165.7 pcf) for the arkose (red sandstone) and 2.54 BSSD (158.5 pcf) for the aphanite which occurs in a limited amount in the vicinity of FD-9T.

(3) Swell Tests. Swell tests on the shale rock core indicates a lineal expansion normal to the foliation of 0.4 of a percent in approximately 160 hours. A typical plot of the test results is shown in Appendix D, Figure D-1.

(4) Moisture. All physical property tests on the rock were conducted at the natural moisture content. All test samples of shale were waxed in the field to retain their natural moisture and were stripped immediately prior to testing. Absorption averaged 1.18 percent for 27 samples in the shales and sandstones and .566 for 15 samples of the basalt. Samples with open joints were tested in a wetted state to simulate the weakest load conditions. No pore pressure measurements were made during physical testing due to the low porosity of the rock and the lack of total confinement during testing.

(5) Modulus of Elasticity and Poisson's Ratios. Elastic moduli and Poisson's ratios were determined as a by-product of the controlled testing and are indicated on Table 1 entitled, "Summary of Test Results" in Appendix D. Laboratory test results will be correlated with in situ tests for the modulus of elasticity to be done using overcoring techniques and tunnel test section during construction. Present considerations indicate design values for the modulus of elasticity should be 2.1 x 10^6 psi for the sedimentary rocks and 4.6 x 10^6 for the basalt. These values are the averages of all the static test results obtained in the testing program. Utilization of the modulus of elasticity E from the dynamic sedimentary rocks and a lower one for the basalt. For the purpose of design it is recommended that the static values for both Poisson"s ration and the modulus of elasticity be utilized for design. A summary of the dynamic and static test results is shown on Table 1 in Appendix D. All values for determining the elastic moduli and the Poisson's ratio were obtained by extrapolating the curves.

(6) Sliding Tests. Maximum normal stresses used were at 5, 10 and 15 TSF on all sliding tests. These values representing a range of values within which the greatest proportion of the loads will fall. Cohesion values are the results of projection of the slopes of the shear envelope to the "0" normal stress intercept in accordance with ETL 1110-2-63 dated 25 June 1969.

b. Sliding Friction. Friction tests on natural joint surface were conducted on open bedding joints in the shale. The average angle of sliding friction for the joint surfaces in a wet condition was 31.6 degrees with an average cohesion of 2.2 TSF primarily due to undulations in the shale surface. A typical plot of the actual test is shown in Appendix D, Figure D-2. The apparent cohesion due to an undulating surface is highly variable depending on shale structure. As there was little evidence of shearing of the surface after testing, it is considered to be more applicable to use a zero cohesion configuration of the sliding envelope for design purposes. This analysis is being verified by conducting tests on sawed surfaces. These natural planes of weaknesses are the most prevalent rock structure throughout the sedimentary types in the tunnel and provide the principle shaping and load factor in excavations. This property will also be a determining cost factor depending on the elected method and direction of tunneling.

c. Triaxial Tests. Triaxial tests are incomplete. Values will be available for the final design.

d. Multistage Triaxial Tests. These tests were conducted on steeply dipping natural joints which cross the natural bedding in the sedimentary rocks. This condition of test was used to evaluate the coefficieint of friction and cohesion along natural strike, dip and diagonal joints planes at varying loads. These joints which intersect the tunnel at random angles to the alignment provide guidance in determining the load configuration on the tunnel support. Due to the irregular joint surfaces the strength values in the sedimentary rock types are generally high resulting in average values for friction of 30.5° degrees and cohesion of 97 TSF. A typical multistage triaxial test is shown on Figure D-8 and the test results are shown on Tables D-1 of Appendix D.

e. Unconfined Compression Tests. Unconfined compression tests were conducted with and without strain measurements. A typical individual test result and a table showing the test results are shown on Figures D-3 through D-7 and Table D-1 in Appendix D. Three design factors have been determined from the measured strain results and the values vary considerably with the rock type. Tests completed to date indicate the following average results for the major rock types:

	Average Compressive Strength	Average Poisson's Ratio (U)	Average Modulus of Elasticity (E)
Shale	7,892 psi	.29	2.1×10^6 psi
Basalt	10,263	.38	4.6×10^6 psi

These test values indicate that the use of machine type tunneling may be feasible from the standpoint of rock strengths. The relative high average strengths of the rock also indicate that there will be relatively small deformation of the tunnel walls in the sound rock areas during the unsupported phase of construction.

f. Dynamic Testing. Selected samples prepared for static testing were sonic tested at varying loads for shear and compression wave velocities. From this testing the dynamic properties of the elastic and shear modulus and Poisson's ratio were derived. This control type testing was used to evaluate the design factors derived from static tests. A comparison of those test results shown on Table D-1 in Appendix D indicate good correlation between tests for similar rock types and condition. This more rapid non-destructive and less expensive testing allows a better sample distribution for the variety of conditions to be encountered during tunneling and may provide better test control during actual construction of the tunnel. Compressive wave testing also verified the inplace field values obtained during the seismic refraction survey and assisted in interpretations of the rock line in the interbedded sandstone and shale layers.

g. Slaking. Slaking tests are being conducted on representative samples of rock types. Particular consideration has been given to the massive shale varieties which have exhibited stress relief characteristics. All tests are in accordance with CRD-C-148-69. Observations of bedrock surface exposures indicate that slaking will not be a major factor in the tunnel construction. Stress relief in the form of sample crazing when immersed in water after air drying has been noted in several shale samples. Total fracturing of the sample takes place in a 3/4 to 1-inch pattern as is shown in Figure D-9 in Appendix D. This condition was noted in a low percentage of the samples and is not considered to be a major factor during tunnel construction when relief stresses will be confined by the tunnel walls. The associated spalling is considered to be within the control of a shotcrete program.

17. HYDROLOGY. The groundwater surface has been measured at all explorations during drilling. Representative boreholes have been used to monitor long term fluctuations of the groundwater surface either with piezometers or as open boreholes. See Table C-1 in Appendix C. Prior to installations of observation wells, borings were pressure tested in 5-foot zones; representative large diameter borings were pump tested to determine a stabilized in-flow capacity. Additional pressure testing is being conducted and multilevel piezometers will be installed to obtain a true hydrostatic pressure gradient at tunnel grade. Observation wells have indicated a rapidly fluctuating water table in the overburden which probably is not representative of the actual hydrostatic condition at tunnel grade. The results of the additional testing will be incorporated in the final design. Present results of pressure testing utilizing procedures recommended in the draft of EM 1110-2-2901 dated December 1973 indicates that water inflow during tunneling will be low averaging less than 3 gpm per lineal foot of untreated tunnel for the major portion of the tunnel alignment (See Figure 3, Appendix E). The required shotcrete surface is expected to reduce this value to a minimal value but the effectiveness of the water control will depend to a large degree on the method of tunneling selected for the contract. Results of pressure testing are shown on the geologic log profiles Plates 9A-5 through 9A-19 and on the individual boring logs. Pressure pumping is continuing, however, preliminary results indicate a relatively low average yield of 11 gpm at a drawdown averaging 33 feet. Pump test results are shown on Table C-2 in Appendix C. Based on these results the drawdown of the water surface above the tunnel is expected to be minimal and mitigated by normal longitudinal recharge from the area. Further analysis will depend on the results of the multilevel piezometers presently being installed.

18. GEOLOGIC FACTORS AFFECTING TUNNELING. The tunnel shape and load configuration is dependent upon the regional conditions of bedding dip and rock structure as depicted on Plates 9A-5 through 9A-19. Anticipated load factors for the rock conditions and tunneling methods are shown on Table 2 in Paragraph 15c. Alternate methods of determining load factors which vary according to the degree of sophistication of the input data are also indicated for consideration in evaluating the most practical support system. Rock conditions have been grouped into three categories for the purpose of design as shown on Table 2. Actual conditions

are expected to exceed the best average condition in most of the tunnel. In the poorer zones no tunneling condition is expected to exceed the load requirements defined by the fault zones. The length of these zones do not have a finite extent although the rock structure indicates that the fault zones do not have great lateral extent normal to the fault line.

a. Tunneling Methods. The method of tunneling will affect to a large extent the tunnel condition and permanent rock loads. As is indicated in Table 2, load factors due to rock loosened during tunneling may be reduced by as much as 50% in the major portion of the tunnel with machine excavation.

b. Standup Time. Standup time will also depend largely on the controls exercised during excavation; however, using an analysis outlined in a report by Bieniawski entitled "Engineering Classification of Jointed Rock Masses," the following standup times are determined for the three rock conditions shown in Table 2.

Best average condition	6 months for 12-foot span
Worst average condition	1 week for 9-foot span
Fault zone	5 hours for 4.5-foot span

This condition is defined by two parameters which consider an unsupported span of rock and the time that it takes this span to fail. The active unsupported span is defined as either the width of the tunnel or the distance from the support to the face of the tunnel if this is less than the tunnel width.

c. Jointing. The high strength irregular surface joints are expected to have a minimal effect on the tunnel configuration with regard to either load or shape. The frequency and orientation of these joints have been indicated by the joint rosettes for some borings shown on Plates 9A-5 through 9A-19 and have been given consideration in the Barton, Lien and Lund analyses (See Appendix E where these strength factors have a strong influence on rock loading.

19. TUNNEL DESIGN FEATURES.

a. General. Three load configurations have been determined for the rock structure and the rock loads determined as shown in Table 2. The more sophisticated method of determining rock loads by Barton, Lein & Lunde based on actual test data values has been included to determine if any ultimate savings in design is represented by the more detailed analysis method. When reduced loading has been assumed for a machine type excavation, the load factors have been reduced in accordance with support factors described in Table 3-3 of draft EM 1110-2-2901.

b. Excavation Conditions. Excavation conditions will be largely determined by the construction method elected. Primary control for water inflow and slaking for conventional excavation methods will be provided by shotcrete without mesh, the thickness varying with the ground conditions. No shotcrete is anticipated if the construction is by machine excavation with precast lining. The grouted lining will provide the necessary control for reducing water inflow and any spalling within a short distance of the heading, not to exceed the stand-up time. The direction of excavation will be a significant factor in the excavation and support difficulties. Although the normal direction would be upgrade and updip (in regard to rock structure) to allow a dry heading, tunneling downgrade under a previously supported roof could improve tunneling conditions and excavation rates. This method may be given increased considerations due to the relatively low water inflow indicated by pressure and pump test data. Assumed overbreak has been based on review of the rock structure for the particular sections and their fracture characteristic for conventional excavation. (See Table 2).

c. Support. Recommended support based on geologic considerations has been indicated in Table 2. These recommended support methods allow for maximum flexibility within short sections where they may be modified to treat conditions as they exist on a per shift basis. Computations are shown in Appendix E for the three ground conditions. The basic design is based on recommendations of the modified Terzaghi method as outlined in the draft EM 1110-2-2901 with resulting loads as indicated in Table 2.

d. Lining. The lining x-section is determined by structural analysis of such governing factors as required permanent rock load support and hydrostatic water pressures. Further analysis of hydrostatic water pressures may reduce the height of the natural groundwater surface above the tunnel crown. This factor will be further analyzed based on the results of piezometers presently being installed. Grouting of the lining appears limited by design to that necessary to close voids resulting from overbreak at the tunnel crown for the conventional method of excavation. Peripheral grouting will be continuously required for the precast type lining design. The grouting of the precast lining will be continuous and follow immediate to the lining installation.

e. Treatment of Water. A special treatment of structures will be required in limited highly broken fault zones where higher water inflows may require panning and diversion of water prior to shotcreting. Steel support spacing will be further adjusted as necessary to meet the existing load conditions. Treatment of water inflow by shotcrete application is recommended to prevent or reduce drawdown of the water table beneath surface structures above the tunnel. Further control will be exercised during construction by monitoring shallow observation wells in the area of critical structures and providing alternate methods of recharging the water table as necessary to prevent unnecessary settlement. Control of water inflow of critical areas by grouting ahead of and during the tunnel construction will allow further reduction of any localized areas of high water inflow.

f. Vibration Controls. Controls will be exercised throughout the work to prevent overstressing any of the surface structures. Some urban structures may require a preconstruction survey to determine safe vibration levels for that particular structure. Construction controls would then be exercised to prevent safe vibration levels from being exceeded during the prosecution of the work.

g. Tunnel Instrumentation. This phase of the project is primarily divided into three phases. Phase I will address the preconstruction determination of existing stree magnitudes and directions along the prosed tunnel center line. Phase II will cover the monitoring of the excavation during construction and the verification of data obtained in Phase I. Phase III will consist of post-construction monitoring of the completed project. The work anticipated to be accomplished in each phase is as follows:

(1) Phase I. This work will consist of in situ stress measurements made by overcoring or similar techniques from the ground surface. The effectiveness of the first installation will determine the need for or the practicality of a second installation. A second test to be conducted, if practical, in the same exploration would be the field testing of a mechanical fracturing device to measure in situ stresses. The device is presently being built for the Waterways Experiment Station. Present plans require that the cost of this testing would be jointly shared by Waterways Experiment Station, U.S. Department of Transportation and New England Division.

(2) Phase II. This work will be carried out under the construction contract and is presently scheduled to be a tunnel section approximately 100 feet in length and consist of three instrumented stations containing borehole extensometers, pore pressure transducers, load cells, convergence points and strain meters. In addition, a microseismic survey would be conducted along each tunnel wall. The purpose of this section would be to evaluate the reliability of the predesign test data and modify the tunnel construction and design to take advantage of any actual proven conditions in the remaining portion to be constructed.

This proposal is in line with the recent suggestion for tunnel test sections as recommended in the paper by K.S. Lane entitled "Field Test Sections Save Costs in Tunnel Support" and as discussed in "Conference on Application of Rock Mechanics to Tunnel Design." Reference Paragraph 23.a(6).

(3) Phase III. This monitoring work would be of limited extent and conducted through readouts from the ground surface. The purpose would be to conduct the studies for 3 to 6 months or until such time as the tunnel is filled, the ground arch stabilizes and the permanent stress is transferred to the lining.

20. CONDITIONS OF SPECIAL ENGINEERING SIGNIFICANCE.

a. Geologic Conditions. Geologic conditions at tunnel grade appear suitable for the use of mechanical excavation equipment incorporated with precast tunnel lining sections. The utilization of such systems would allow for immediate installation of the tunnel thus reducing the period of the tunnel serving as a drain beneath the city. Machine excavation would have lesser vibrations than those caused by conventional drill and blast methods.

b. Hydrologic Conditions. Hydrologic conditions may result in relatively low inflow of water during excavation. To a large extent this may allow the tunnel Contractor to proceed downslope with a minimum of pumping. This condition could improve the tunnel support conditions and result in reduced temporary support through driving down the prevailing dip of the rock structure.

c. Special Support Requirements. A relatively unstable condition of the rock is predicted between Stations 93+50 and 95+50 in the tunnel intake area. Due to the critical unstable nature of surface slopes in this area a maximum amount of steel support has been allowed. This area may also require the necessity of pregrouting before excavations to reduce uncontrolled inflow and settlement of the ground surface.

21. RECOMMENDATIONS. Based on the detailed geologic study and available tests data it is recommended that consideration be given to utilizing tunnel design and load factors provided under the more sophisticated systems of Barton, Lien and Lunde. The Terzaghi classification (1946) while dominant in the USA for over 25 years and excellent for the purpose for which it was evolved is basically applicable to tunnels with steel supports and is not as suitable for modern tunneling methods using shotcrete and rock bolts. Calculations of a series of load configurations for this design indicates that the Terzaghi classification generally results in overdesign of rock conditions not requiring steel for temporary support. All factors assumed for design purposes will be verified by a tunnel instrumented section during construction.

D. STRUCTURAL DESIGN CRITERIA

22. PURPOSE AND SCOPE. This section presents the design criteria, basic data and assumptions made in the structural design of the Auxiliary Conduit Tunnel. Typical design computations are included in Appendix F.

23. DESIGN CRITERIA.

a. General. Allowable stresses, loading conditions, design assumptions and other criteria were based on applicable parts of the following references. Detailed criteria is given for each of the major construction materials.

(1) Working Stresses for Structural Design, EM 1110-1-2101, 1 November 1963.
(2) Engineering and Design, Tunnels and Shafts in Rock, EM 1110-2-2901, Draft December 1973, plus revised update of 5 November 1976.
(3) Standard Practice for Concrete, EM 1110-2-2000, 1 November 1971.
(4) Details of Reinforcement - Hydraulic Structures, EM 1110-2-2103, 21 May 1971.
(5) Conduits, Culverts and Pipes, EM 1110-2-2902, 3 March 1969.
(6) Memorandum, "Conference on Applications of Rock Mechanics to Tunnel Design", held at New England Division on 28 and 29 January 1976, and first indorsement DAEN-CWE-G dated 15 March 1976.
(7) Rock Mechanics in Engineering Practice", Stagg and Zienkewicz John Wiley and Sons, Ltd., Publishers.

b. Concrete. Concrete working stresses are in general accordance with ACI Standard Building Code Requirements for Reinforced Concrete (ACI 318.63) as modified by EM 1110-1-2101. Concrete with a minimum ultimate compressive strength of 4,000 pounds per square inch was selected for design of the cast-in-place liners as recommended in EM 2902. Concrete with a minimum ultimate compressive strength of 5,000 pounds per square inch was selected for design of the precast liner. Following is a listing of maximum working stresses for the concrete as used in design of the tunnel liner.

	fc' = 4000 P.S.I.	fc' = 5000 P.S.I.
Flexure (extreme fiber in compression)	1800 P.S.I.	2250 P.S.I.
Flexure (extreme fiber in tension)	102 P.S.I.	113 P.S.I.
Average Compressive Stress	1000 P.S.I.	1250 P.S.I.
Principal Diagonal Tensile Stress	126 P.S.I.	141 P.S.I.

c. Steel Reinforcement.

(1) General. Details of reinforcement will be in accordance with EM 1110-2-2103, Details of Reinforcement - Hydraulic Structures.

(2) Grade and Working Stress. All reinforcement is designed for a working stress of 20,000 P.S.I. in tension. The reinforcement will be deformed bars made of new billet steel, intermediate grade (ASTM A-615, Grade 40), conforming to Federal Specification QQ-S-632C, Type II, Class B40.

(3) Spacing. The clear spacing between parallel bars will not be less than 1-1/2 times the nominal diameter of the bars except that in no case will the clear distance between parallel bars be less than 1-inch or 1-1/2 times the maximum size of the coarse aggregate.

(4) Minimum Cover for Reinforcement. A minimum clear distance of 3 inches will be specified between the inside face of the liner and the inside ring of main transverse reinforcing. A minimum clear distance of 2" will be specified between the outside transverse reinforcing and the shotcrete surface for cast-in-place liner. A minimum clear distance of one inch will be specified for the outside transverse reinforcement in the precast liner.

(5) Splices. Lap splices will be in accordance with EM 1110-2-2103 and the ACI Building Code. Tension lap splices will be staggered longitudinally so that no more than one half the bars are spliced at any section within the required lap length. Splices will not be made at points of maximum stress. No reinforcement bars larger than No. 11 will be required.

(6) Temperature and Shrinkage Reinforcement. Longitudinal temperature and shrinkage reinforcement will be provided in the tunnel liner. The reinforcement will provide for a minimum ratio of steel to design concrete area of .0025 at the inside face of the cast-in-place liner. The same percentage of longitudinal reinforcement will be provided as distribution steel and in the precast lining and will be split between the inside and outside faces.

(7) Minimum Flexural Reinforcement. Where transverse reinforcement is specified the amount specified will provide for minimum ratio of steel to design concrete area of .0025. Transverse reinforcement on the inside face of the liner will be continuous in the cast-in-place liners.

d. Structural Steel. Full circle ring beams will be ASTM A36 steel. They have been proportioned on the basis of a maximum allowable fibre stress of 26,200 p.s.i. (0.73 fy) as provided in EM 1110-1-2101 for temporary loading.

e. Shotcrete. Shotcrete will be specified in conjunction with both rock bolt and steel ring beam rock support systems. The shotcrete will prevent slaking of the rock face, control seepage, stabilize rock fragments and provide supplementary lateral support for the steel ring beams. Generally the shotcrete will be applied to the entire rock surface above the tunnel springline. It can be applied before rock support systems are installed where immediate seepage control is required and where it is safe to do so. It will also be applied below the springline for control of water in limited reaches with large inflows. Temporary drains will provide relief through the shotcrete coating to control seepage during application until sufficient strength has been obtained. Shotcrete thickness of 1, 2 and 3 inches will be specified depending on the rock condition encountered. The 1-inch thickness will be used with the nominal rock bolt pattern, a 2-inch thickness with the heavy rock bolt pattern and 3 inches in conjunction with steel ring beam supports. Wire mesh reinforcement of the shotcrete is not considered necessary for thicknesses specified and use intended. Prior to application the rock surface will be cleaned by water jet. The required thickness will be built up in two layers and will be applied by the dry mix process. Shotcrete strengths of 500 p.s.i. and 3,000 p.s.i. at 8 hours and 28 days respectively will be specified.

f. Rock Bolts. Rock reinforcement will consist of resin type rock bolts 11 feet in length at a maximum spacing of 4.5 feet on center. The length of bolt is set in accordance with Table 3-7 Item C, 3 of draft EM 1110-2-2901. The anchors will be No. 10 reinforcing bars with a threaded end, bearing plate and nut. Tensioning the bolts after installation is not considered necessary as long as the plate is bearing on the rock. A minimum pull out strength of 30,000 pounds will be specified and field test will be required before a proposed bolt is approved for production bolting.

g. Increase in Normal Working Stress. A one-third increase in the allowable stresses has been permitted in the permanent concrete lining for the construction and maintenance condition. This condition assumes full external water pressure with the tunnel empty combined with rock loading.

h. Control Joints. The cast-in-place tunnel lining will be placed in 30-foot long monoliths. The control joints between placements will have continuous plastic or rubber waterstops. The longitudinal reinforcement in the liner will be continuous through the joints.

(2) Analysis and Design. The precast liner is analyzed as an eight segment circular member connected by pins incapable of transmitting momemt between segments. The liner is assumed to interact with the surrounding rock. Two orientations of segment joints were investigated for the various loading conditions. One position assumes a segment spanning the crown and the second position has the joints rotated 22.5 degrees with a joint occurring at the crown. The joint position spanning the crown governed design in all cases.

An ultimate concrete strength of 5,000 psi will be specified for the segments to be used in all reaches of the tunnel. Details of the segments and their reinforcement are shown on Plate 9A-22. Stresses in the liner segments for the various conditions of loading are tabulated in Appendix F.

d. External Water Pressure. The level of groundwater in the vicinity of the conduit will vary from approximately 150 to 180 feet above the tunnel invert. The groundwater level generally varies with the ground surface decreasing to its minimum level at the river. It is assumed that pressure on the tunnel liner from external water will always exist after construction regardless of the permeability of the rock mass. Measurements of hydrostatic pressure in the rock will be made both prior to and during construction as an aid in planning control measures. The buoyancy effect resulting from the difference in external pressure between the crown and invert causes bending in addition to the normal thrust on the liner cross section and has been considered.

e. Internal Water Pressure. The auxiliary conduit tunnel will be completely full when in operation. The internal pressure on the tunnel lining will be determined by level of water at the intake and outlet shafts. During peak flows in the Park River the intake end of the tunnel will be subjected to a maximum internal pressure equal to the maximum hydraulic gradient in the junction structure. The internal pressure at the outlet end will be determined by the level of the Connecticut River at the outlet shaft. The ground area over the tunnel is protected from river flooding by a system of floodwalls and dikes thereby allowing the pressure inside the tunnel to rise with the river level above the level of ground water outside the tunnel. A maximum net internal pressure equivalent to a height of water of 30 feet will occur at the location of minimum ground water elevation when the river is at maximum flood stage. According to analysis discussed later in this memorandum, this pressurized condition will not require a steel lining on the inside surface of the concrete liner. The possibility of hydrostatic pressures in the rock being substantially below the pressures which would result from observed ground water levels has been considered. Such an occurrence would result in higher internal pressures than discussed above. However, it is not likely that long reaches of the rock will exist without intercepting water bearing joints. The pressure of this water once introduced will always exist between the rock face and the concrete liner at least to the level of normal river water. If a lag should exist between an increase in river stage and hydrostatic pressure in the rock it would most probably occur in the more competent rock (rock with rock quality designation greater than 65) which would offer greater restraint (higher modulus of deformation) to the liner than assumed for design. It is concluded therefore that the 30 feet of differential head discussed above represents the maximum internal pressure for which the finished tunnel liner should be designed.

f. Rock Loads. Rock loading on the roof of the tunnel was determined from the geologic investigations discussed earlier. For purposes of determining temporary support systems and designing the permanent liner, three rock conditions were considered. The rock loadings have been classified for ease of reference by the type of temporary support required by each.

The best average rock condition is referred to as that requiring nominal rock bolt support and is assumed to exert a vertical loading of 0.5 T.S.F. (approximately one-quarter a tunnel diameter of rock) on the tunnel roof. This condition is the most prevalent of the three and is expected along approximately 88% of the tunnel length. The second condition of rock, worst average, that requiring heavy rock bolt support, is assumed to produce a rock load of 1.1 T.S.F. and exists along a reach of tunnel approximately 800 feet in length between Stations 23+00 and 31+00 as indicated on the geologic profiles. The third rock condition is that requiring structural steel sets for a rock loading of 2.4 T.S.F. and is that condition expected to be encountered in the fault zones in the vicinity of stations 57+50 and 95+50. The rock loads noted above are those used in design of the cast-in-place liner with drill and blast excavation. For design of the liners with mole excavation rock loadings have been reduced to 0.25 T.S.F., 0.7 T.S.F. and 1.75 T.S.F. for the nominal rock bolt, heavy rock bolt and structural steel set rock support systems respectively. These reductions in load reflect the advantage gained from machine excavation in accordance with Table 2 in Section C, Geotechnical Design. The rock loading assumptions above, derived from rock core observations and seismic investigations, will be verified by or corrected for results of instrumentation installed in the test tunnel section during construction.

g. Grouting Pressure. A pressure of 15 p.s.i. was assumed for contact grouting in design of the tunnel liner. This pressure applied to precast liners will insure the liner remains in compression under net internal head conditions. Grouting pressure on the precast liner will be applied to the full ring to control stresses which would occur under partial loading.

h. Excavation. Excavation for the Auxiliary Conduit will be either by convential drill and blast or by tunneling machine (mole). Excavation by tunneling machine will be required if the precast permanent liner alternative is selected. For the cast-in-place drill and blast alternate a minimum line of excavation and a payment line for excavation has been established, The minimum line of excavation, the "A" line, is that line beyond which no unexcavated material shall remain. The "B" line is established as that line within which measurement for payment of excavation will be made. The distance between the "A" and "B" has been varied with the type of temporary rock support required and represents a reasonable estimate of the average overbreak expected in the three rock conditions as defined in this memorandum. A theoretical volume of shotcrete, computed on the basis of tunnel length, "B" line diameter and required thickness of shotcrete will be deducted from the concrete quantitites for payment. The volume of concrete measured for payment will be that lying between the inside liner surface and the "B" line reduced as noted above. For the machine tunneling alternatives, excavation will be required within a minimum diameter which will provide an annular space between the outside of the liner and the excavated rock surface. The annular space will allow for back packing and grouting in the case of a precast liner and for temporary rock support systems for the cast-in-place liner. No "A" or "B" lines will be established for the machine tunneling alternatives and excavation will be paid on the basis of price bid per linear foot of tunnel.

i. Rock Modulus. For purposes of analyzing stresses in the concrete liners a foundation modulus of 1,000 K.C.F. (580 P.C.I.) for the rock was assumed.

E. PREPARATION OF DESIGN COMPUTATIONS

25. GENERAL. The tunnel lining and steel supports were analyzed by computer utilizing the program "Tunnel," as described in Technical Report C-73-2, "Computer Study of Steel Tunnel Supports" by U.S. Army Engineers Waterway Experiment Station. The program uses the stiffness matrix method of solution and provides for the interaction of rock and supports. A support modulus is computed, relating the liner or support stiffness to the foundation modulus for the rock, and given as program input together with geometry, section properties and loading. Representative printouts of computer input and results are included in Appendix F for the cast-in-place drill and blast and the precast liner alternatives.

26. TEMPORARY ROCK SUPPORT.

a. General. Temporary rock support systems have been selected for the three rock conditions described previously and details of each for the drill and blast alternative have been shown on Plate 9A-21. The tunnel reaches along which use of each of the systems is anticipated for design purposes are indicated on Plates 9A-5 through 9A-19. The limits of use of each system of support can be altered during construction when actual rock conditions are discovered. Temporary rock supports will be required only if a cast-in-place alternate is selected and will provide the primary rock support for periods of up to one year prior to placement of the permanent lining. The precast alternate, if selected, will be installed immediately behind the tunnel heading. Timely backpacking and grouting will preclude the need for temporary support. A thin application of shotcrete may be required for the precast alternate in very limited areas of short stand-up time or when delays in lining operations are encountered.

b. Steel Ring Beams. Steel ring beams will be specified for temporary rock support in the fault zones which intercept the tunnel alignment. The beams have been designed to support the vertical rock loadings of 2.4 TSF and 4.8 TSF at longitudinal spacings of 4 feet and 2 feet respectively. The beams will be full circular and will most probably be fabricated in four segments. Splices between segments will consist of bolted webs and welded flanges. Longitudinal bracing between ring beams will be required at a maximum spacing of five feet. Additional lateral support will be provided by the shotcrete coating required in conjunction with ring beams. The rock loadings input to the computer were derived from resolution of vertical loadings into radial and tangential forces. The tangential force was limited to that value produced by friction equivalent to an angle of 25 degrees with the horizontal. The resolution of the radial forces into vertical and horizontal components provided the loadings for the computer analysis. Blocking points were assumed to be at 48 inches on center. Blocks were assumed to be 12-inch thick oak with a surface area of 8 inches by 10 inches in contact with ring beam flanges, for the purpose of computing a support modulus. The controlling stresses in the ring beams occur at the crown of the tunnel, where maximum moment is coincident with minimum axial force from the rock loading. An 8W67 beam was selected for all areas where structural steel supports are required. Variation in actual rock conditions encountered during construction will be accommodated by varying the longitudinal spacing of beams, holding a maximum spacing of 4 feet wherever their use is directed. The 3-inch thickness of shotcrete to be specified will make the use of lagging unnecessary.

c. Rock. The spacings and lenghth of rock bolts have been determined based on the tunnel span. Although no initial prestress of the rock face is considered to be necessary under the present design the diameter of the bolts (1-1/8-inch) was selected based on its ability to develop a passive force of 6.3 psi normal to the rock face, with the maximum spacing of bolts, at a stress not exceeding three quarters of its yield strength. A yield strength of 50,000 psi was assumed for design and the specifications will require that the bolts be field tested to this stress prior to final determination of bolt size and type. Bolting patterns developed for the cast-in-place drill and blast alternative are shown on Plate 9A-21.

d. Shotcrete. Shotcrete coatings specified for the tunnel are intended, as previously stated, to prevent slaking, control of water and for safety of personnel. They are used with adequate temporary rock support systems and will not function as primary support. No attempt was made at a rational design. The thicknesses given, 1, 2 and 3 inches for the respective support systems, were selected empirically after review of available literature including Report No. S-76-4, "State of the Art Review of Shotcrete," prepared for the office of Chief of Engineers.

27. PERMANENT LINING.

a. General. The permanent linings, cast-in-place and precast have been designed to withstand all loadings. Rock support systems are considered to be effective in limiting loss in strength of the rock mass and the loadings selected for the liners reflect this. The steel ring beams are considered to act as reinforcement steel when cast into the permanent liner. The minimum design thickness of 13 inches for the cast-in-place lining was selected to provide a clear space of 9 inches at the crown for placement of concrete by slick pipe. Specific loading conditions, analysis and design procedures used for each of the alternate linings are discussed below.

b. Cast-in-Place Liner.

(1) Loading Conditions. The loading conditions considered in design of the cast-in-place tunnel liner are as follows:

Condition 1 - Full external water head, including internal water pressure
Condition 2 - Full rock load, no water
Condition 3 - Full external water and rock load combined including internal water pressure
Condition 4 - Grout pressure only
Condition 5 - Construction and maintenance condition; full external water head and rock load with no internal water pressure combined with allowable stresses increased by one third.

The weight of the liner is included for all loading conditions. For loading Condition 1 - external and internal water heads of 150 and 180 feet above the invert respectively, were assumed. This combination of water heads resulted in the maximum axial tensile force on the liner cross section, occurring at Station 28, and when combined with rock loading in Condition 3 becomes the critical condition for design of tensile reinforcement steel in the liner. An alternate combination of water heads was assumed for loading Condition 1 resulting in a net external head of 85 feet. This net external head when combined with rock loading in Condition 3 produces the maximum fibre stress in the liner for comparison to allowable stress without increase. Maximum average compressive stresses are produced by loading Condition 5. Loading Conditions 2 and 4 did not produce critical stress conditions in the liner. The average and maximum compressive stresses and stresses in reinforcement steel under the various loading conditions are tabulated in Appendix F.

(2) Analysis and Design. The computer analysis referred to above assumes the cast-in-place liner to be a continuous ring interacting with the surrounding rock. Stresses in the liner were checked at critical locations for the combinations of moments and axial load produced by the various conditions of loading. The transformed area method of computation was used in computing stresses in cracked sections. Thicknesses of 13 and 19 inches were assumed for design of the liner with rock bolt and structural steel supports respectively. This discounts the thickness of allowable shotcrete encroachment inside the "A" line. The tensile stress in the concrete liner will be below the allowable tensile stress for concrete alone under a net internal head of 30 feet for loading Condition 1. No tensile cracking of the concrete liner will result therefore, and steel lining of the concrete liner is not necessary to control leakage from the tunnel during maximum pressure conditions. The amount of transverse reinforcement at the crown varies with the rock loading and analysis indicates No. 7 bars at 12 inches, No. 9 bars at 12 inches and No. 8 bars at 6 inches are required for the 0.5 TSF, 1.1 TSF and 2 TSF rock loadings respectively for the drill and blast alternate. Transverse reinforcement will be continuous around the full circumference of the liner on the inside face. The area of the ring beams is utilized as concrete reinforcement in the heavy rock loaded area. Printouts of computer analysis for the various loading conditions and design computations for the cast-in-place drill and blast alternative have been included in Appendix F. Loading, moment and thrust diagrams have been shown on Plate 9A-20. Rock loadings shown for loading conditions 2 and 3 and 5 are those for the drill and blast excavated nominal rock bolt supported sections.

c. Precast Liner.

(1) Loading Conditions. The loading conditions for the precast liner are the same as those outlined for cast-in-place liner. The rock loadings for mole excavation have been used for design of the precast segments as explained in the paragraph on rock loads. The net internal pressure case of loading Condition 1 is not valid for analysis of the precast liner due to its inability to carry axial tensile forces. Grouting pressures (Case 4) around the circumference of the liner will be utilized to insure the liner segments remain in compression under this condition.

(2) Analysis and Design. The precast liner is analyzed as an eight segment circular member connected by pins incapable of transmitting momemt between segments. The liner is assumed to interact with the surrounding rock. Two orientations of segment joints were investigated for the various loading conditions. One position assumes a segment spanning the crown and the second position has the joints rotated 22.5 degrees with a joint occurring at the crown. The joint position spanning the crown governed design in all cases.

An ultimate concrete strength of 5,000 psi will be specified for the segments to be used in all reaches of the tunnel. Details of the segments and their reinforcement are shown on Plate 9A-22. Stresses in the liner segments for the various conditions of loading are tabulated in Appendix F.

F. CONCRETE MATERIALS

28. CONCRETE MATERIALS. The basic concrete materials investigation set forth in Design Memorandum No. 4 is applicable for the total project and therefore for both phases of construction. Design Memorandum No. 4 and its indorsements provide all information required by EM 1110-2-2000 "Standard Practice for Concrete" for both phases of construction and no updating of information has been found necessary. Design Memorandum No. 4 and its indorsements states that fine and coarse aggregates from three commercial sources in the area be listed as approved sources in the specifications for each of the two construction phases of subject project. This is still the case and these three approved sources will be listed in project specifications for Phase II construction. One of the three sources, The Balf Company, has been selected by the Contractor for use in Phase I construction. The Phase II project will require a concrete plant with a production capacity of 65 cubic yards per hour, which is well under the capacities of the commercial plants operating in the area. Concrete will be centrally mixed and delivered to the project site and placed in hoppers connected to a drop pipe connecting from ground elevation to tunnel elevation. Drop pipes will be approximately twelve inches in diameter and located at the inlet and outlet shafts and also at each of the two storm drain shafts. Using these four locations the maximum placing distance from a drop shaft would be approximately 2,000 feet. Concrete will be collected from the drop pipe by an agitating hopper at tunnel elevation and then transferred and placed by pump, or conveyed to a pump for placement by high speed conveyer belts or rail cars with screw type feeders. If the precast liner alternate is selected, concrete aggregates used in the production of the precast units must be from one of the three approved aggregate sources, or, as in the case of cast-in-place concrete, from a source that is equal or better in quality than that of the poorest of the three approved sources. Specifications for the precast segments will require that cements and other applicable controls be the same as those required for cast-in-place concrete as stated in Design Memorandum No. 4. Materials for use in shotcreting shall be required to conform to requirements of ASTM-C-33 and can be supplied by any of the nearby commercial sources of aggregate.

G. CONSTRUCTION SCHEDULE

29. CONSTRUCTION SCHEDULE.

a. General. The Box Conduit feature and the Auxiliary Conduit feature are sufficiently different and independent such that construction can be initiated independently. Construction of the Box Conduit was initiated in July 1976 and will run for a period of 4 construction seasons (3-1/2 years). The Auxiliary Conduit would commence approximately 1 year later than the box conduit construction start and would run for a period of 3 years. The schedule as set forth herein is considered reasonable and one that takes into account economics.

b. Auxiliary Conduit. It is assumed that the contract will be awarded in spring of 1977, the 2nd construction season. The phases of construction are briefly outlined below, whereas the details of construction were more fully discussed in Section H, "Construction Procedure and Diversion Plan" of GDM No. 2, Phase II, Part II - Auxiliary Conduit.

The construction schedule is based upon the assumption that the contractor will construct the tunnel by the conventional drill and blast method with a cast-in-place liner. This appears to be the most economical means of construction for this particular project. If the contractor elects to use a tunnel boring machine with either a precast or cast-in-place liner, the construction sequences are expected to vary from those stated. The overall construction schedule, however, is not expected to be altered significantly.

(1) Second Construction Season (First Year Auxiliary Conduit Construction). The Contractor is expected to concentrate on the construction of the river shaft. It will take the Contractor approximately 6 months to prepare the site, construct and lower the caisson into place, grout-seal the interface between caisson and bedrock and complete the mucking out of the caisson. An additional 2 months will be required to continue the vertical shaft through rock down to EL-147 msl and an additional month will be required to set up his horizontal tunneling equipment. The Contractor will then proceed with his tunneling operation from the river shaft driving upward to the Pope Park end. The tunneling operation is expected to proceed uninterrupted throughout the winter. The tunneling heading should advance at a rate of approximately 750 ft./month based upon round-the-clock operations, 6 days a week. As the rock excavation progresses the rock anchors are to be installed. By the end of the 12th month (3rd month of tunneling) the Contractor is expected to be at Sta. 30+00.

(2) Third Construction Season (Second Year Auxiliary Conduit Construction).

(a) Tunneling. The Contractor is expected to continue tunneling to the intake shaft at Pope Park. By the end of the second year the tunneling operation is expected to be complete.

(b) River Shaft. Once the river shaft is no longer needed for rock removal the Contractor is expected to proceed to modify the river shaft. At the junction between the tunnel and the river shaft, rock will have to be carefully removed to provide the necessary curvature for the bend; this is to be followed by the installation of the concrete lining both in the tunnel and in the shaft. At the same time the Contractor is expected to construct the single-wall braced steel sheet cofferdam around the outfall structure (top of river shaft).

(c) Intake Shaft. Towards the middle of the 2nd construction season, the Contractor is expected to initiate construction at the intake structure, which includes the installation of the earth support system. During the winter months he will remove the overburden and excavate the necessary rock down to the tunnel invert and initiate the installation of the concrete liner.

(3) 4th Construction Season (Third Year Auxiliary Conduit Construction).

(a) Tunnel. Installation of the concrete liner is to be completed.

(b) Intake Shaft. The Contractor will form and place the concrete, backfill, remove the earth support system, if removal is required, and perform final grading, landscaping and cleanup.

(c) River Shaft. The Contractor is expected to excavate to grade, remove the segment of the caisson above El. -3.0 msl, form and place the necessary concrete, remove the cofferdam and excavate and place the rock paving. The necessary grading and cleanup would complete the river phase of the project.

H. COST ESTIMATE

30. COST ESTIMATE.

a. Conduit Extension. The contract for the first half of the local protection project was advertised and the contract awarded to Vicon Construction Company of New Jersey for the bid price of $23,300,000. The bid price was approximately $5,282,000 less than the Government estimate of $28,582,155.

b. Auxiliary Conduit. The previous detailed cost estimate of $38,390,000 for the Auxiliary Conduit was included in Design Memorandum No. 2, Phase II, Part II, Auxiliary Conduit dated 24 January 1975. The estimate was based upon a price level of November 1974. For comparison purposes, the estimate has been updated to the November 1976 price level by increasing the cost by 18.6% (per ENR). The cost, updated, would be $45,530,000.

Detail cost estimates for the three alternate means of construction (conventional w/ cast-in-place, mole w/ cast-in-place and mole w/ precast liner) have been prepared. The conventional system of construction with the cast-in-place liner was found to be the least expensive $35,500,000 (TABLE H-1). The costs of mole excavation with precast segments at $36,000,000 (TABLE H-2) and mole excavation with cast-in-place liner at $39,425,000 were found to be more expensive.

The latest cost of the project, $35,500,000 is approximately 10 ~~8~~,030,000 cheaper than the updated GDM estimate. The reduction in cost is due to the more advance design of the tunnel itself as well as the design of the shafts. Major cost savings were attributed to the reduction of liner thickness in certain reaches, the elimination of the cleanout shaft and the reduction of contingencies from 20% to 15%.

TABLE H-1

DETAILED COST ESTIMATE
(NOVEMBER 1976 PRICE LEVEL)

Conventional Drill & Blast w/ Cast in Place Liner

Description	Estimated Quantity	Unit	Unit Price	Estimated Amount
Mobilization	1	Job	L.S.	$ 100,000
Maint. & Control of Traffic	1	Job	L.S.	125,000
Control of Water	1	Job	L.S.	1,365,000
Excavation				
Earth, common	68,000	C.Y.	4.50	306,000
Rock, tunnel	176,000	C.Y.	60.00	10,560,000
Rock, shafts	4,600	C.Y.	40.00	184,000
Fills				
Gravel	5,000	C.Y.	5.00	25,000
Random	40,000	C.Y.	2.50	100,000
Stone Protection				
Temporary	1,000	C.Y.	30.00	30,000
Permanent	1,100	C.Y.	25.00	27,500
Tunnel Support Steel				
Rock Anchors	13,000	EA.	54.00	702,000
Ring Beams	335,000	LBS.	0.60	201,000
Reinforcing Steel	3,585,000	LBS.	0.40	1,434,000
Concrete				
Tunnel Lining	46,000	C.Y.	130.00	5,980,000
Structural	5,100	C.Y.	100.00	510,000
Mass	2,000	C.Y.	60.00	120,000
Cement	350,000	CWT.	2.70	945,000
Grout, tunnel	18,000	C.F.	14.00	252,000
Shotcrete	72,000	C.F.	8.00	576,000
Concrete Caisson	1	Job	L.S.	1,650,000
Cofferdam	1	Job	L.S.	700,000
4'Ø Drainage Shafts	1	Job	L.S.	180,000
Earth Support System	1	Job	L.S.	280,000
Stoplogs	1	Job	L.S.	35,000
Clean-out and Dewatering Provisions	1	Job	L.S.	100,000

Description	Estimated Quantity	Unit	Unit Price	Estimated Amount
Waterstops	28,000	L.F.	7.00	$ 196,000
Misc. Metals	155,000	LBS.	1.10	170,500
Chain Link Fencing	325	L.F.	10.00	3,250
Gate-Chain Link, 20' Double Swing	1	EA.	600.00	600
3" Bit Conc. Pavement	5,000	S.Y.	5.00	25,000
6" Topsoil seeded	20,000	S.Y.	2.00	40,000
	Sub-Total			$26,922,850
	Contingencies (15%)			4,027,150
	Construction Cost			30,950,000
	Engineering and Design			2,500,000
	Supervision & Administration			2,050,000
	Total - Auxiliary Conduit			$35,500,000

TABLE H-2

DETAILED COST ESTIMATE
(NOVEMBER 1976 PRICE LEVEL)

Mole w/ Pre-Cast Liner

Description	Estimated Quantity	Unit	Unit Price	Estimated Amount
Mobilization	1	Job	L.S.	$ 3,000,000
Maint. & Control of Traffic	1	Job	L.S.	125,000
Control of Water	1	Job	L.S.	1,365,000
Excavation				
Earth, common	68,000	C.Y.	4.50	306,000
Rock, tunnel	153,000	C.Y.	85.00	13,005,000
Rock, shafts	4,600	C.Y.	40.00	184,000
Fills				
Gravel	5,000	C.Y.	5.00	25,000
Random	40,000	C.Y.	2.50	100,000
Stone Protection				
Temporary	1,000	C.Y.	30.00	30,000
Permanent	1,100	C.Y.	25.00	27,500
Tunnel Support Steel	-0-			-0-
Reinforcing Steel	375,000	LBS.	0.40	150,000
Concrete				
Pre-Cast Lining	9,100	L.F.	530.00	4,823,000
Structural	5,100	C.Y.	100.00	510,000
Mass	2,000	C.Y.	60.00	120,000
Cement	45,000	CWT.	2.70	121,500
Grout, tunnel } liner back packing	16,200	C.F.	14.00	226,800
Pea Stone } liner back packing	1,400	C.Y.	12.00	16,800
Concrete Caisson	1	Job	L.S.	1,650,000
Cofferdam	1	Job	L.S.	700,000
4'Ø Drainage Shafts	1	Job	L.S.	180,000
Earth Support System	1	Job	L.S.	280,000
Stoplogs	1	Job	L.S.	35,000
Clean-out and Dewatering Provisions	1	Job	L.S.	100,000

Description	Estimated Quantity	Unit	Unit Price	Estimated Amount
Misc. Metals	155,000	LBS.	1.10	$ 170,500
Chain Link Fencing	325	L.F.	10.00	3,250
Gate-Chain Link, 20' Double Swing	1	EA.	600.00	600
3" Bit Conc. Pavement	5,000	S.Y.	5.00	25,000
Topsoil, seeded (6")	20,000	S.Y.	2.00	40,000
	Sub-Total			$27,319,950
	Contingencies (15%)			4,105,050
	Construction Cost			31,425,000
	Engineering and Design			2,515,000
	Supervision & Administration			2,060,000
	Total - Auxiliary Conduit			$36,000,000

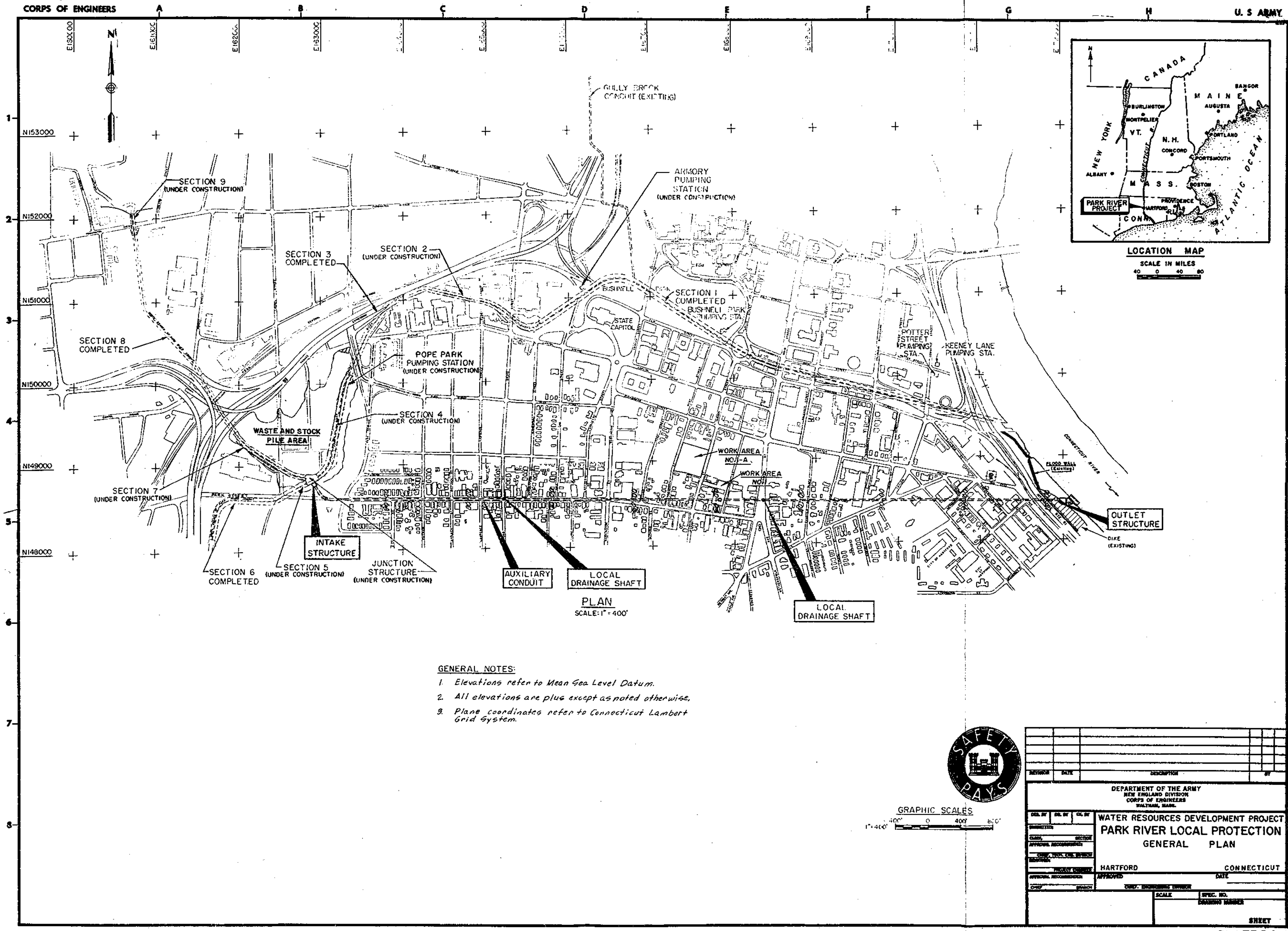
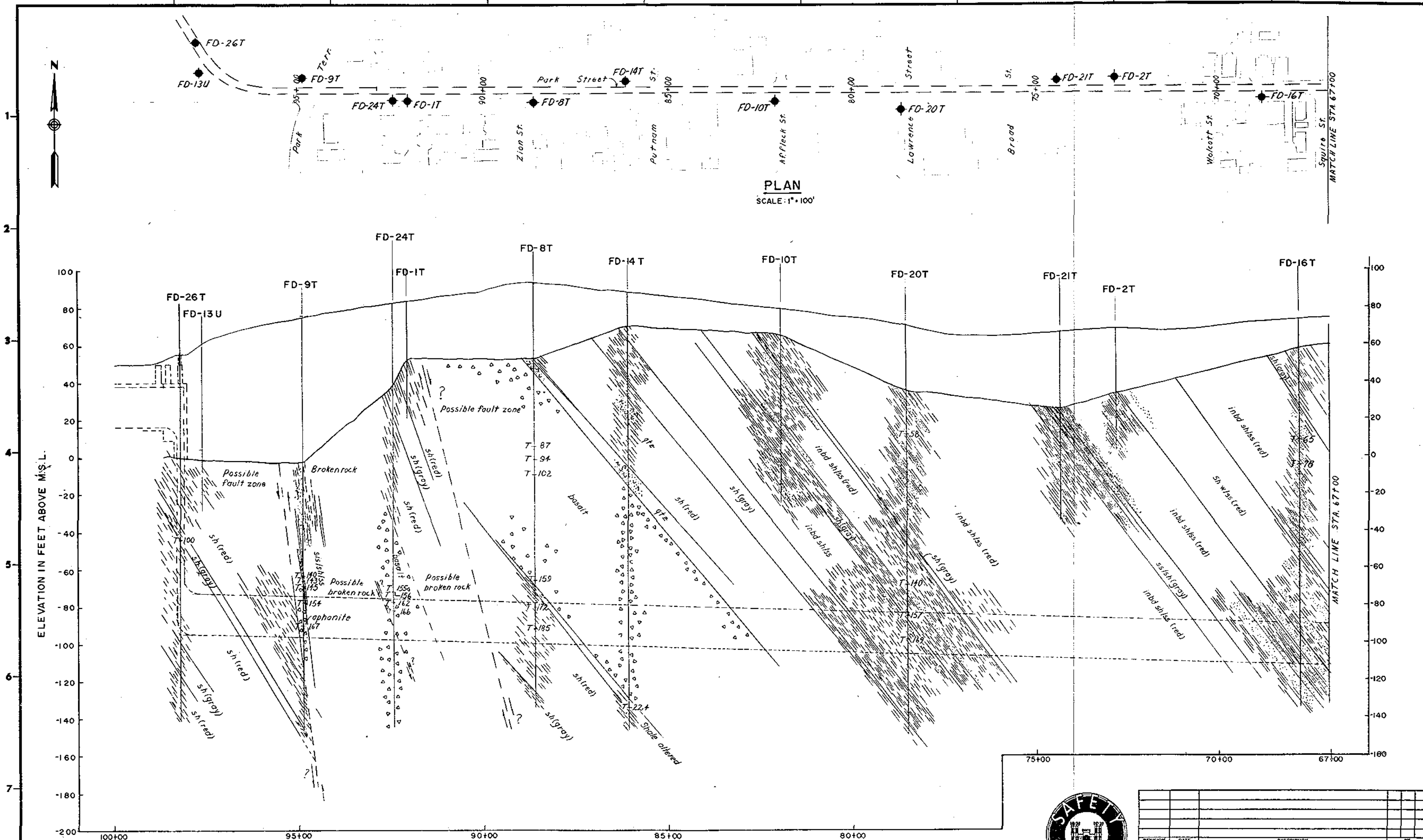

CORPS OF ENGINEERS
U. S. ARMY
GULLY BROOK CONDUIT (EXISTING)
SECTION 9 (UNDER CONSTRUCTION)
ARMORY PUMPING STATION (UNDER CONSTRUCTION)
SECTION 3 COMPLETED
SECTION 2 (UNDER CONSTRUCTION)
SECTION 1 COMPLETED
BUSHNELL PARK PUMPING STA.
STATE CAPITOL
SECTION 8 COMPLETED
POPE PARK PUMPING STATION (UNDER CONSTRUCTION)
POTTER STREET PUMPING STA.
KEENEY LANE PUMPING STA.
SECTION 4 (UNDER CONSTRUCTION)
WASTE AND STOCK PILE AREA
WORK AREA NO. 1-A
WORK AREA NO. 1
FLOOD WALL (Existing)
CONNECTICUT RIVER
SECTION 7 (UNDER CONSTRUCTION)
OUTLET STRUCTURE
DIKE (EXISTING)
INTAKE STRUCTURE
SECTION 6 COMPLETED
SECTION 5 (UNDER CONSTRUCTION)
JUNCTION STRUCTURE (UNDER CONSTRUCTION)
AUXILIARY CONDUIT
LOCAL DRAINAGE SHAFT
LOCAL DRAINAGE SHAFT
PLAN
SCALE: 1" = 400'
N153000
N152000
N151000
N150000
N149000
N148000
CANADA
MAINE
VT.
N.H.
NEW YORK
MASS.
CONN.
ATLANTIC OCEAN
BANGOR
AUGUSTA
BURLINGTON
MONTPELIER
PORTLAND
CONCORD
PORTSMOUTH
ALBANY
BOSTON
PROVIDENCE
HARTFORD
PARK RIVER PROJECT
LOCATION MAP
SCALE IN MILES
40 0 40 80
GENERAL NOTES:
1. Elevations refer to Mean Sea Level Datum.
2. All elevations are plus except as noted otherwise.
3. Plane coordinates refer to Connecticut Lambert Grid System.
SAFETY PAYS
GRAPHIC SCALES
1" = 400'
400' 0 400' 800'
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.
WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
GENERAL PLAN
HARTFORD
CONNECTICUT
SHEET

PLATE 9A-1

CORPS OF ENGINEERS U. S. ARMY



GEOLOGIC PROFILE ALONG ℄ OF TUNNEL

SCALE: HOR. 1" = 100'
VERT. 1" = 20'

LEGEND

SH/SIS	SHALE/SILTSTONE	SDY SH	SANDY SHALE
INBD SH/SS	INTERBEDDED SHALE AND SANDSTONE		BASALT
SS	SANDSTONE	QTZ	QUARTZITE
		T-150	TEST SAMPLE DEPTH

NOTES:

1. ONLY COMPLETED TESTS ARE SHOWN. TESTING PROGRAM IS CONTINUING.
2. TEST RESULTS ARE SHOWN IN TABLES ACCORDING TO BORING NUMBER AND DEPTH.

GRAPHIC SCALES

1" = 100' 100' 0 100' 200'

1" = 20' 20' 0 20' 40'

SAFETY PAYS

REVISION	DATE	DESCRIPTION	BY

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

DES. BY | DR. BY MC | CK. BY

SUBMITTED: CHIEF, SECTION

APPROVAL RECOMMENDED: CHIEF, TECH. ENG. BRANCH

REVIEWED: PROJECT ENGINEER

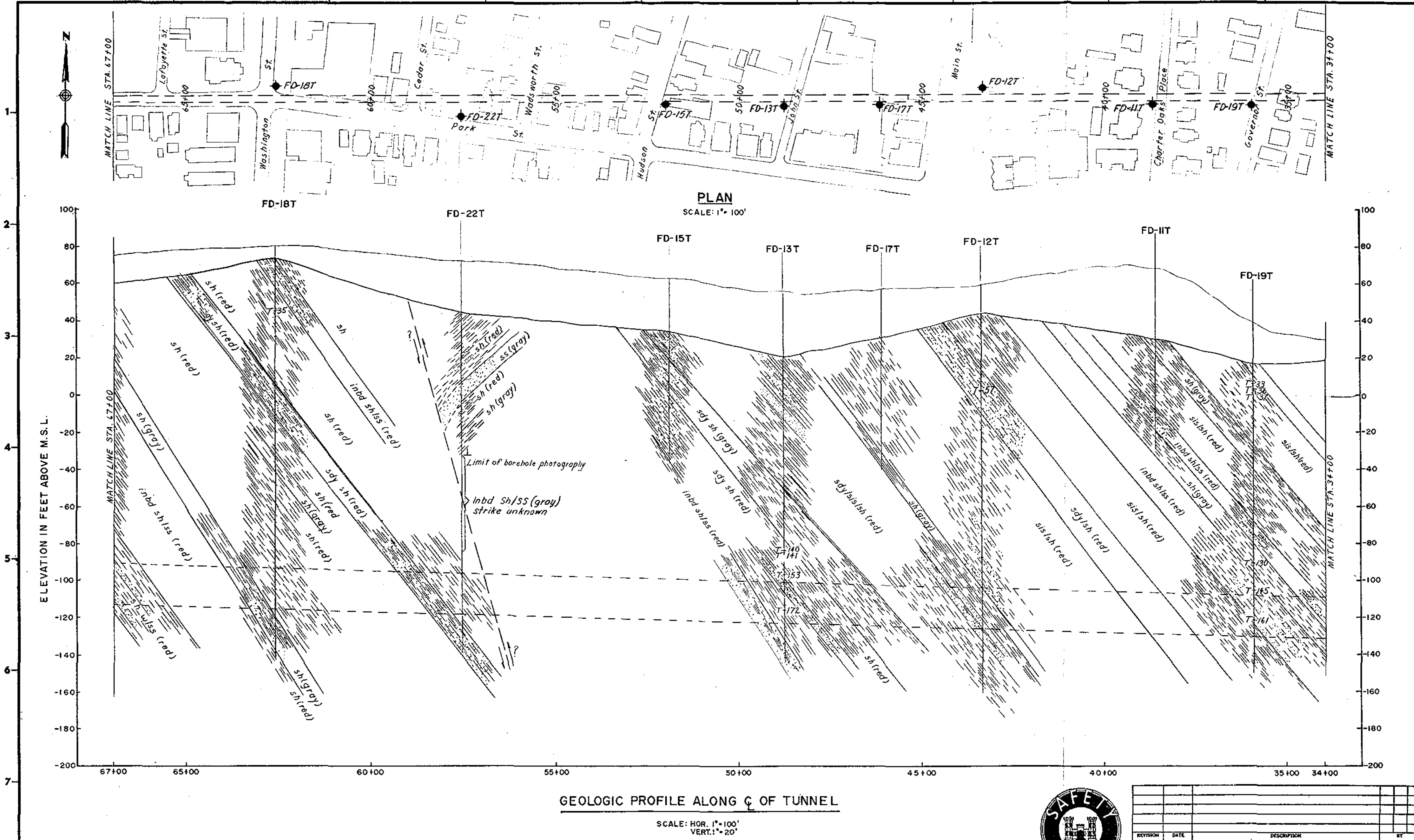
APPROVAL RECOMMENDED: CHIEF BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
AUXILIARY CONDUIT TUNNEL
PLAN AND GEOLOGIC PROFILE
STA. 95+00 - STA. 67+00
HARTFORD CONNECTICUT

APPROVED CHIEF, ENGINEERING DIVISION DATE

SCALE | SPEC. NO. | DRAWING NUMBER | SHEET

PLATE 9A-2



REVISION	DATE	DESCRIPTION	BY

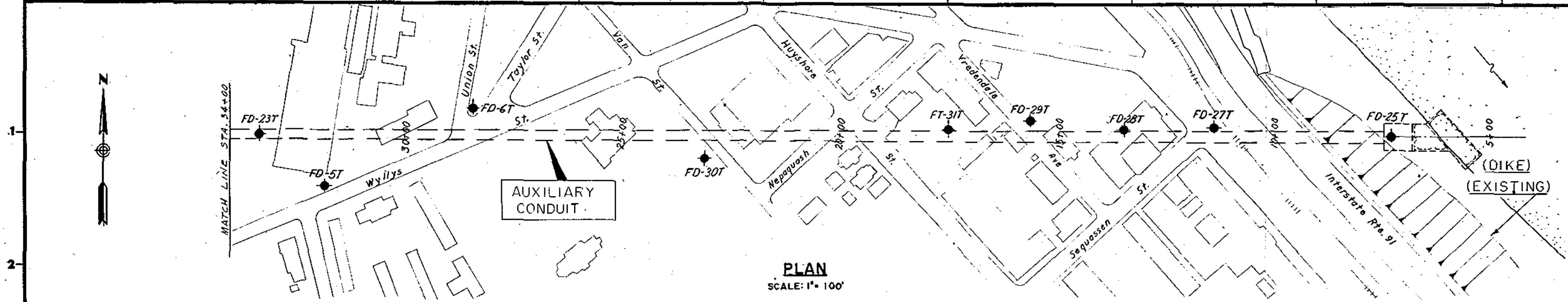
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
AUXILIARY CONDUIT TUNNEL
PLAN AND GEOLOGIC PROFILE
STA. 67+00 - STA. 34+00
HARTFORD CONNECTICUT

DES. BY | DR. BY | CK. BY
SUBMITTED: CHIEF, SECTION
APPROVAL RECOMMENDED: CHIEF, TECH. ENG. BRANCH
REVIEWED: PROJECT ENGINEER
APPROVAL RECOMMENDED: CHIEF BRANCH
APPROVED DATE
CHIEF, ENGINEERING DIVISION
SCALE | SPEC. NO.
DRAWING NUMBER
SHEET

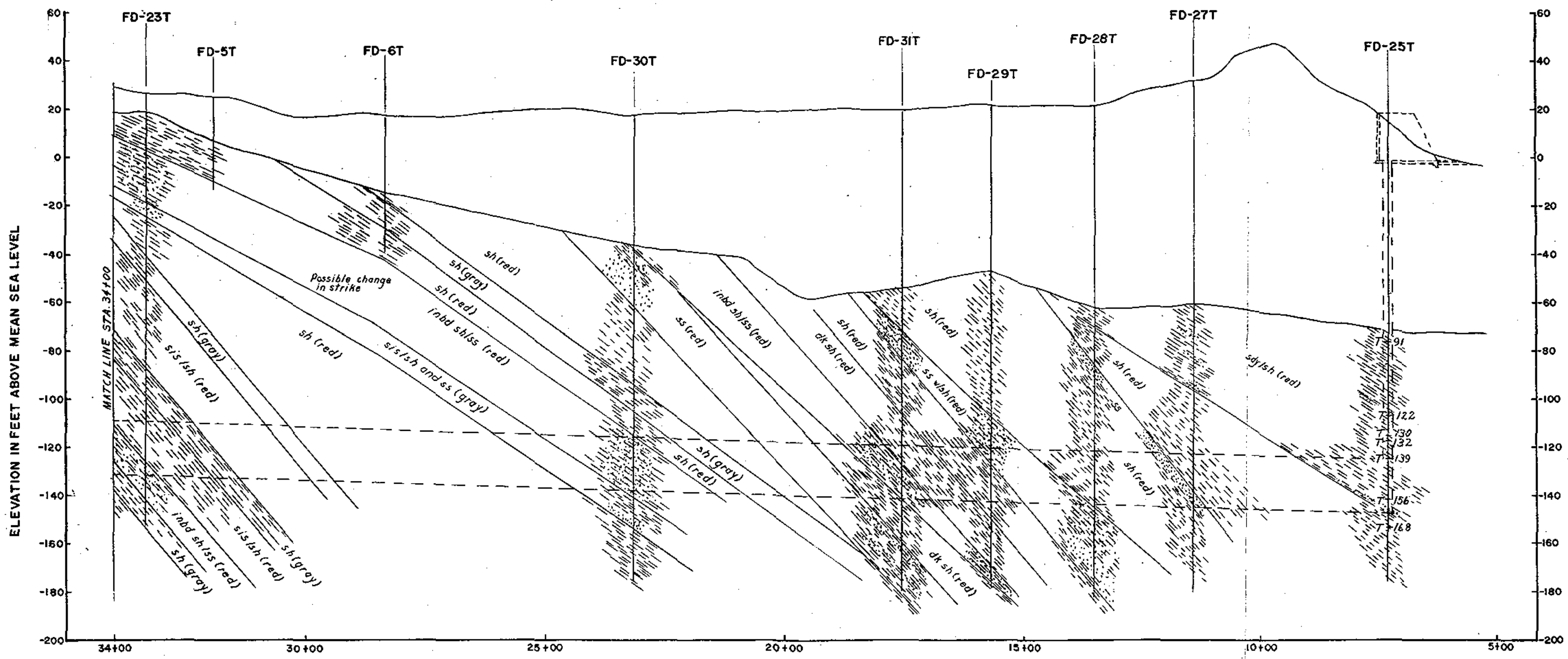
PLATE 9A-3

CORPS OF ENGINEERS U. S. ARMY



PLAN

SCALE: 1" = 100'



GEOLOGIC PROFILE ALONG ℄ OF TUNNEL

SCALE: HOR. 1" = 100'
VERT. 1" = 20'

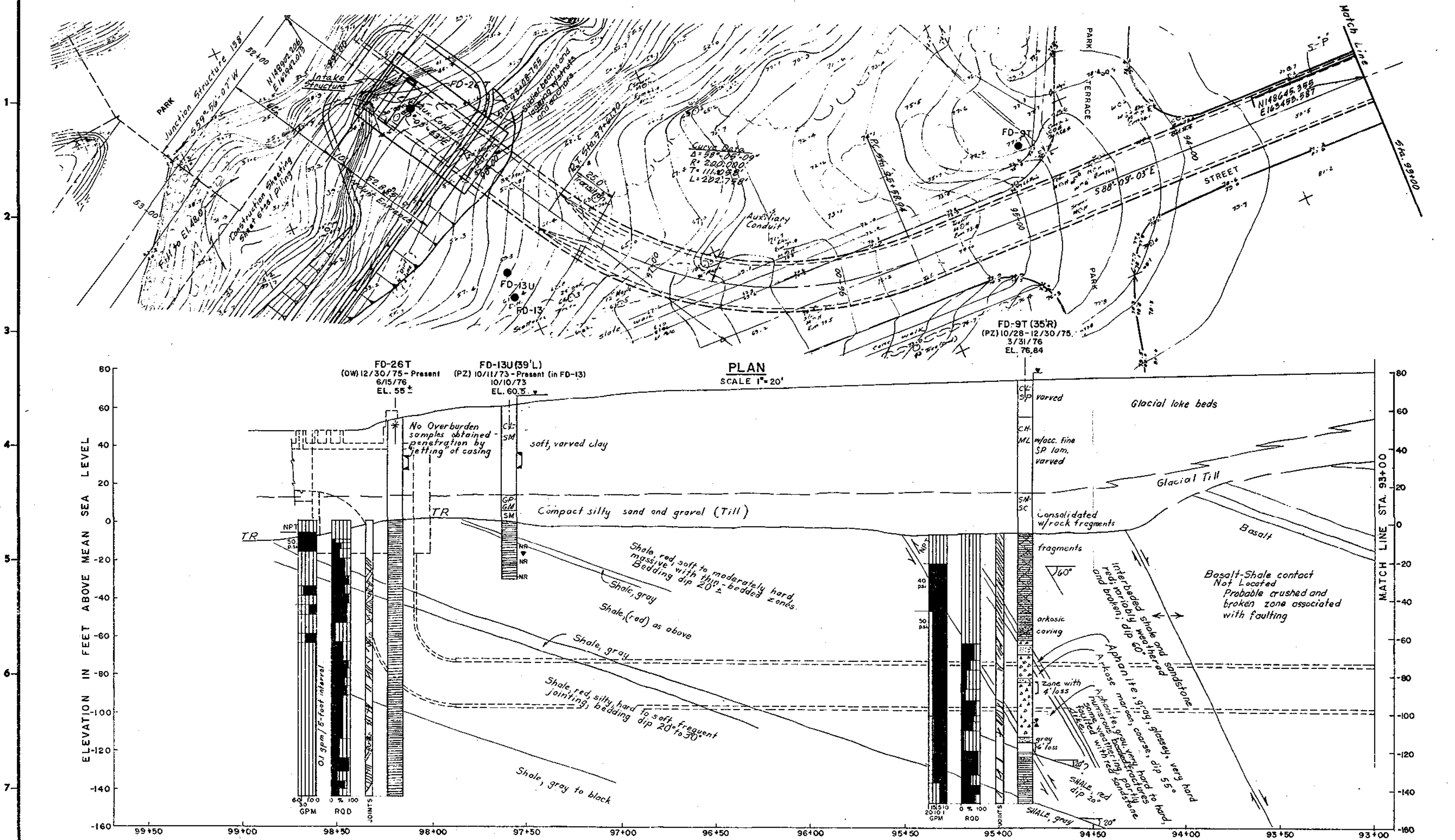
GRAPHIC SCALES
1" = 100' 100' 0 100' 200'
1" = 20' 20' 0 20' 40'

REVISION	DATE	DESCRIPTION	BY

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
AUXILIARY CONDUIT TUNNEL
PLAN AND GEOLOGIC PROFILE
STA.34+00 - STA. 5+00
HARTFORD CONNECTICUT

PLATE 9A-4



GEOLOGIC LOG PROFILE

SCALE 1"= 20'

LEGEND

- SANDSTONE, MODERATELY HARD CALCAREOUS
- INTERBEDDED SANDSTONE AND SHALE
- SHALE, SILTY, SOFT TO MODERATELY HARD
- BASALT AND INTRUSIVE FLOW
- BROKEN ZONE AND/OR BRECCIATED
- FAULT (ASSUMED LOCATION AND DIRECTION OF MOVEMENT)
- S - SEISMIC LINES

LEGEND FOR GRAPHIC LOGS

FD-20T	TYPE AND NUMBER OF EXPLORATION
(OW/PZ) 1-7/15/74	OBSERVATION WELL/PIEZOMETER AND PERIOD OF OBSERVATION
1/7/73	MONTH/DAY/YEAR EXPLORATION COMPLETED
EL. 31±	ELEVATION OF GROUND SURFACE
	MAXIMUM ARTESIAN HEAD
	SUBSURFACE WATER LEVEL
	ARTESIAN FLOW ENCOUNTERED
	RANGE OF SUBSURFACE WATER LEVEL
	BOTTOM OF OBSERVATION WELL/PIEZOMETER
SM	UNIFIED SOIL CLASSIFICATION SYSTEM SYMBOL
NR/NS	NO RECOVERY/ NOT SAMPLED
	ROCK TYPE SYMBOL

*NPT 50 p.s.i. 20 5 1 0 GPM "A" 0 % 100 RQD "B" JOINTS "C"

NOTES:

A. NPT-NO PRESSURE TEST PERFORMED
ASTERISK DENOTES SECTION COULD NOT BE SEALED FOR TESTING
50 P.S.I.-CONSTANTLY MAINTAINED PRESSURE FOR 1 TO 5 MINUTES. VOLUME LOSS IN GALLONS PER MINUTE UNDER CONSTANT PRESSURE. TESTS CONTINUOUS IN 5-FOOT SECTIONS. SCALE EXPANDED FROM 0 TO 1 GPM FOR CLARIFICATION OF LOW PRESSURE LOSSES.

B. ROCK QUALITY DESIGNATION - PERCENT OF CORE PER RUN WITH PIECES LONGER THAN 4 INCHES.

C. DIP AND FREQUENCY OF JOINTS.
"S" INDICATES SLICKEN SIDES ON NATURAL JOINT SURFACE.

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

DES. BY	DR. BY	CK. BY

SUBMITTED:
CHIEF, SECTION
APPROVAL RECOMMENDED:
CHIEF, TECH. ENG. BRANCH
REVIEWED:
PROJECT ENGINEER
APPROVAL RECOMMENDED:
CHIEF BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
AUXILIARY CONDUIT
PLAN AND GEOLOGIC LOG PROFILE
STA. 99+00 TO STA. 93+00
HARTFORD CONNECTICUT

APPROVED DATE
CHIEF, ENGINEERING DIVISION

SCALE | SPEC. NO.
DRAWING NUMBER
SHEET

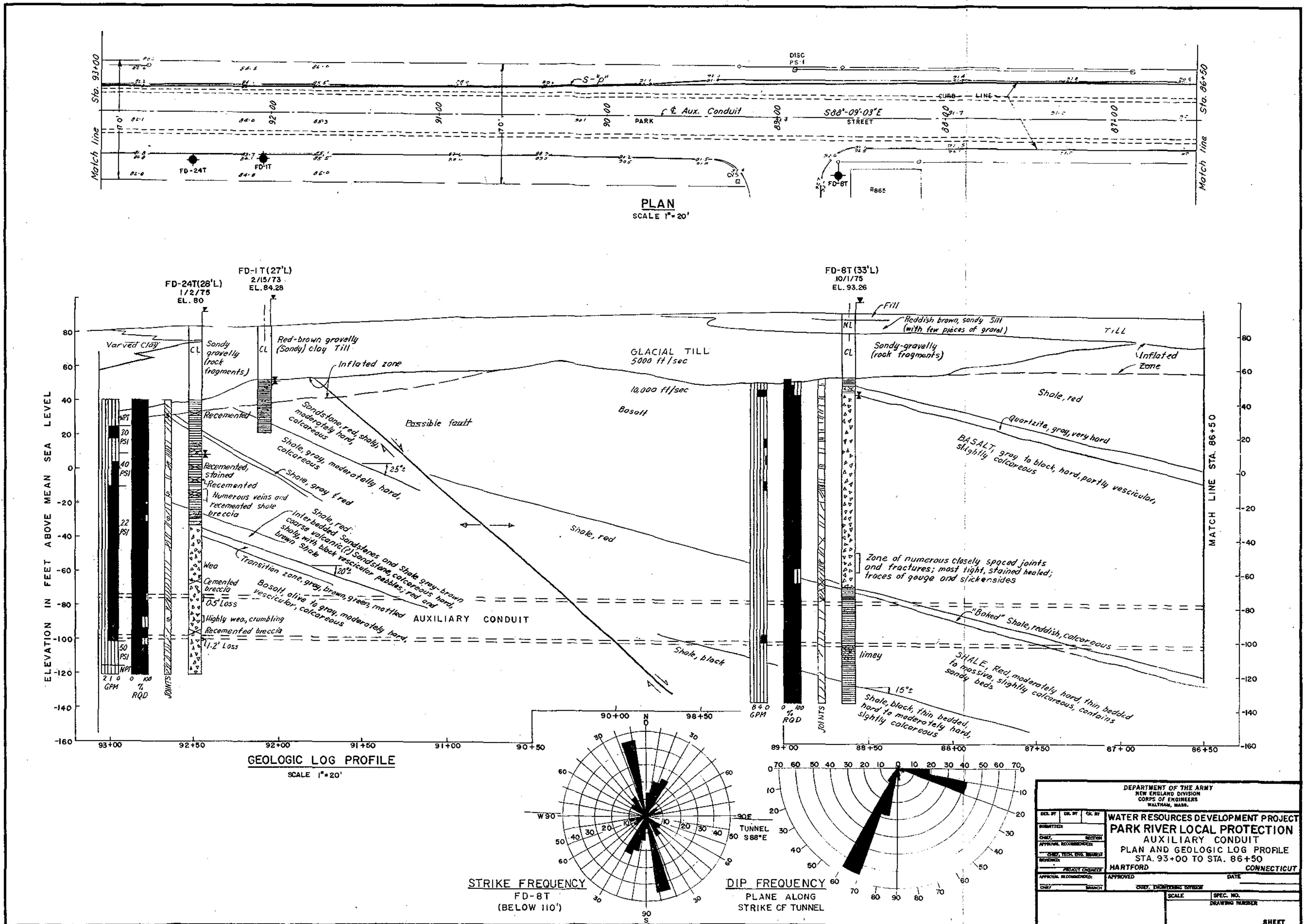
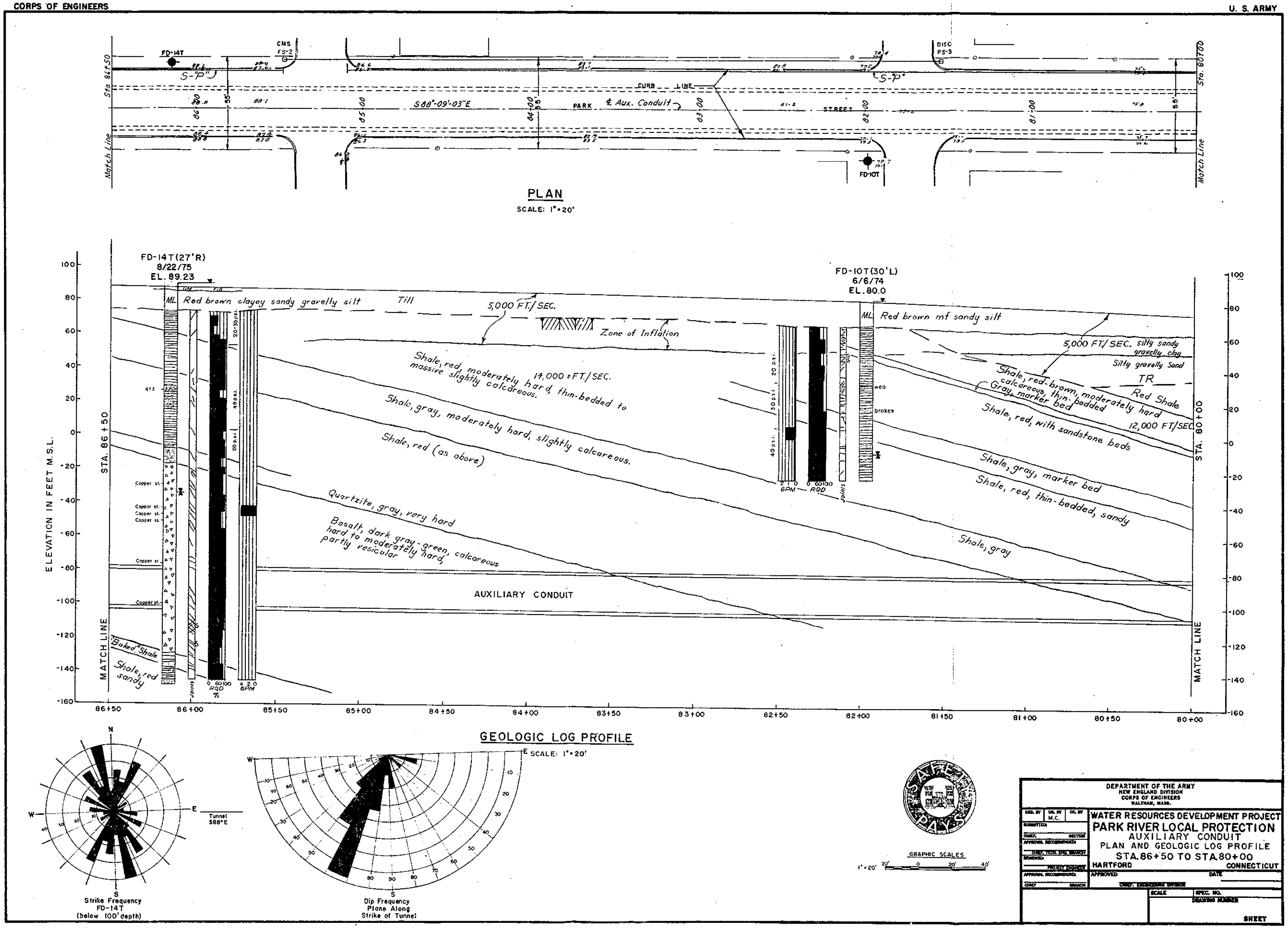
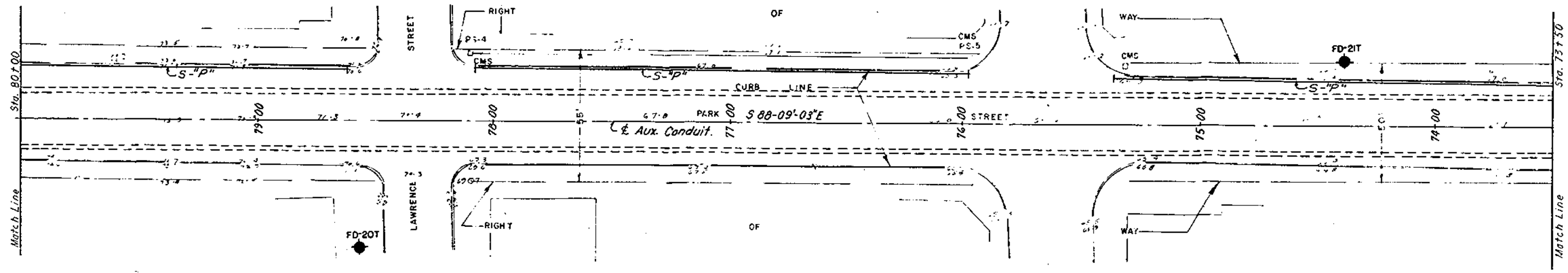

PLAN
SCALE 1"=20'
Sta. 93+00
Match line
Sta. 86+50
Match line
℄ Aux. Conduit
S88°-09'-03"E
PARK
STREET
CURB LINE
DISC PS-1
FD-24T
FD-1T
FD-8T
FD-24T(28'L)
1/2/75
EL. 80
FD-1T(27'L)
2/15/73
EL. 84.28
FD-8T(33'L)
10/1/75
EL. 93.26
ELEVATION IN FEET ABOVE MEAN SEA LEVEL
MATCH LINE STA. 86+50
Varved Clay
Sandy gravelly (rock fragments)
Red-brown gravelly (Sandy) clay Till
Inflated zone
GLACIAL TILL
5000 ft/sec
10,000 ft/sec
Basalt
Fill
Reddish brown, sandy Silt (with few pieces of gravel)
TILL
Sandy-gravelly (rock fragments)
Inflated Zone
Shale, red
Quartzite, gray, very hard
BASALT, gray to black, hard, partly vesicular, slightly calcareous
Possible fault
Sandstone, red, shaly, moderately hard, calcareous
Shale, gray, moderately hard, calcareous
Shale, gray fred
Recemented
Recemented, stained
Recemented
Numerous veins and recemented shale breccia
Shale, red; Interbedded Sandstones and Shale gray-brown coarse volcanic(?) Sandstone, calcareous hard, shaly, with black vesicular pebbles; red and brown Shale
Transition zone, gray, brown, green, mottled
Basalt, olive to gray, moderately hard, vesicular, calcareous
Wea
Cemented breccia
0.5' Loss
Highly wea, crumbling
Recemented breccia
1.2' Loss
AUXILIARY CONDUIT
Shale, red
Shale, black
Zone of numerous closely spaced joints and fractures; most tight, stained healed; traces of gouge and slickensides
"Baked" Shale, reddish, calcareous
limey
SHALE, Red, moderately hard, thin bedded to massive, slightly calcareous, contains sandy beds
Shale, black, thin bedded, hard to moderately hard, slightly calcareous
25°±
20°±
15°±
GPM
% RQD
JOINTS
GEOLOGIC LOG PROFILE
SCALE 1"=20'
STRIKE FREQUENCY
FD-8T
(BELOW 110')
TUNNEL S88°E
DIP FREQUENCY
PLANE ALONG STRIKE OF TUNNEL
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.
WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
AUXILIARY CONDUIT
PLAN AND GEOLOGIC LOG PROFILE
STA. 93+00 TO STA. 86+50
HARTFORD
CONNECTICUT

PLATE 9A-6



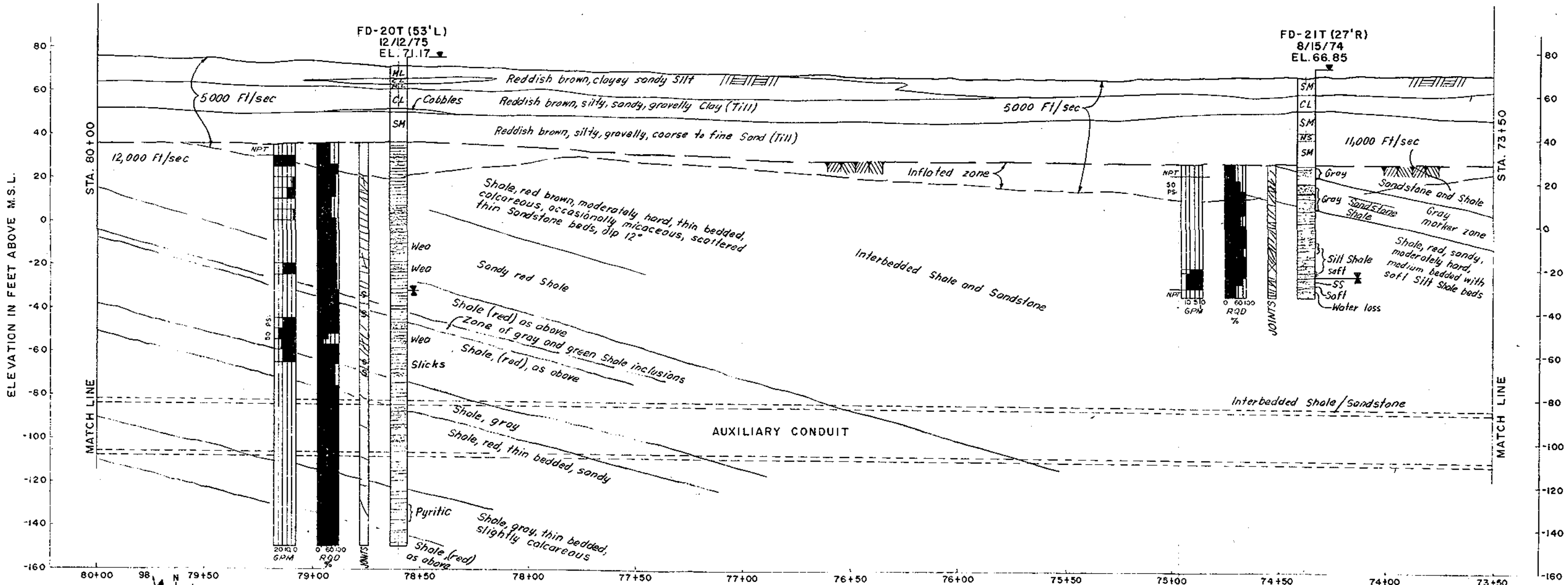
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
AUXILIARY CONDUIT
PLAN AND GEOLOGIC LOG PROFILE
STA. 86+50 TO STA. 80+00
HARTFORD CONNECTICUT



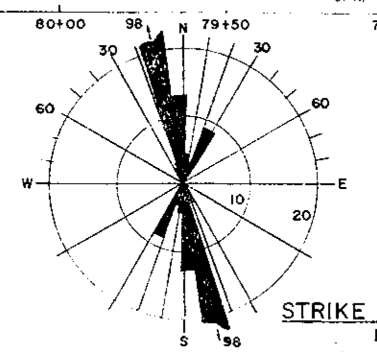
PLAN

SCALE: 1"=20'

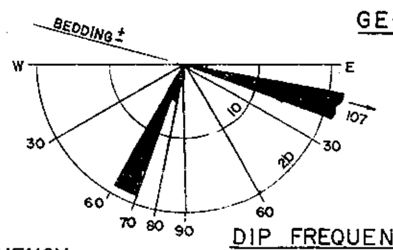


GEOLOGIC LOG PROFILE

SCALE: 1"=20'



STRIKE FREQUENCY
FD-20T
(BELOW 90')



DIP FREQUENCY
PLANE ALONG
STRIKE OF TUNNEL

GRAPHIC SCALES
1" = 20'

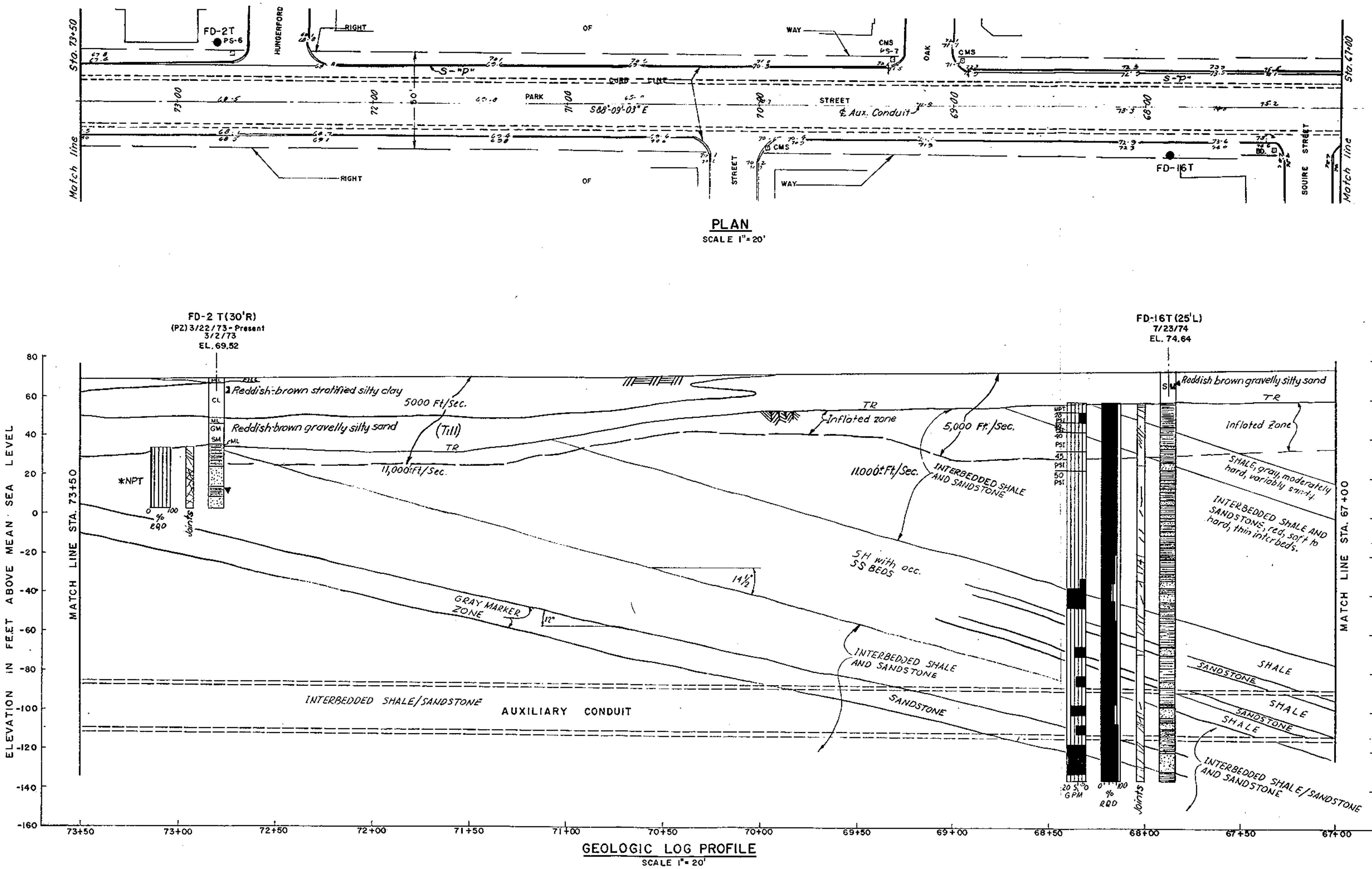
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

DR. BY M.C.

WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
AUXILIARY CONDUIT
PLAN AND GEOLOGIC LOG PROFILE
STA. 80+00 TO STA. 73+50
HARTFORD CONNECTICUT

APPROVED DATE
SCALE SPEC. NO.
DRAWING NUMBER
SHEET

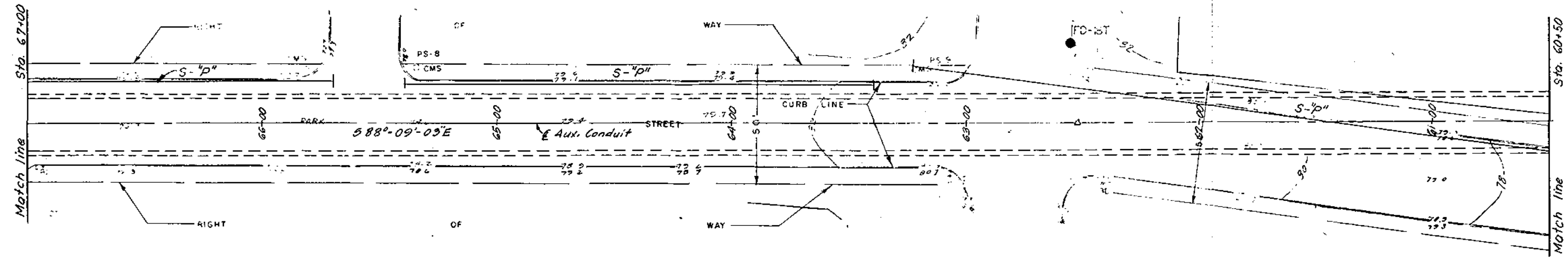
PLATE 9A-8



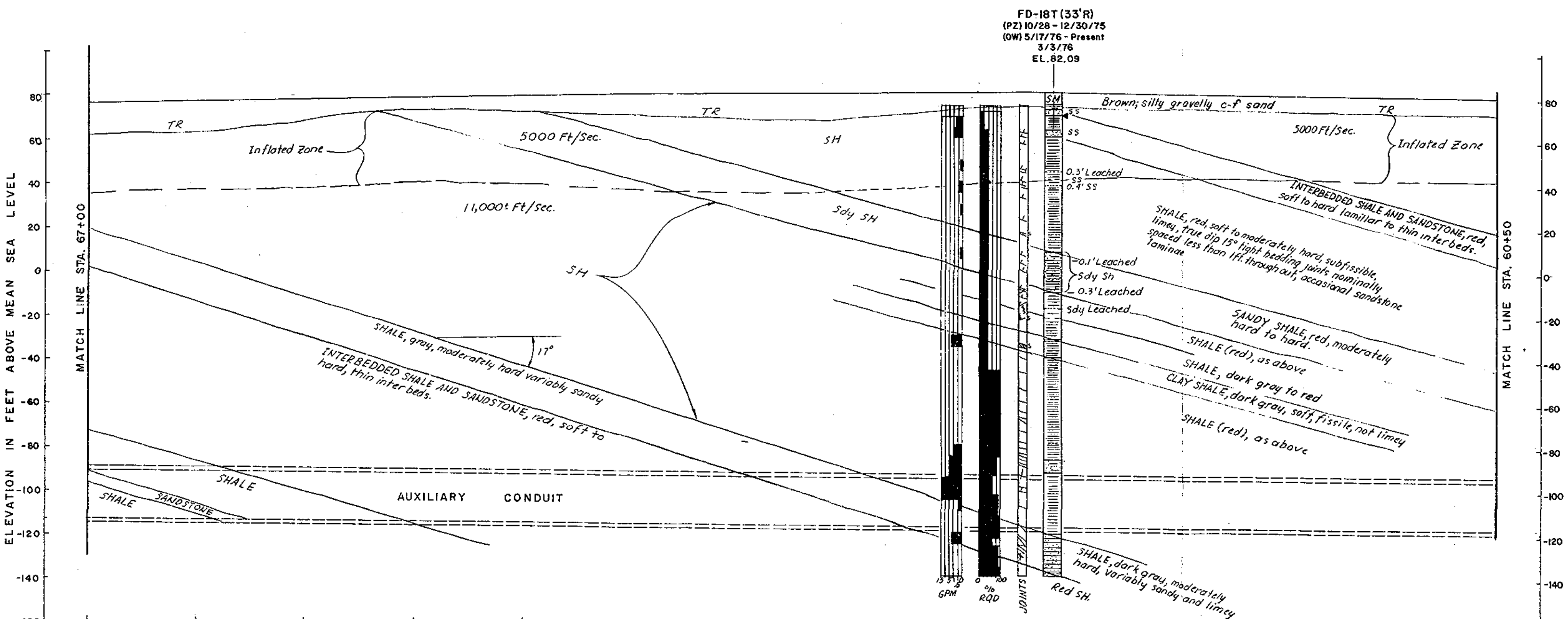
GRAPHIC SCALES
1" = 20' 20' 0 20' 40'

DEPARTMENT OF THE ARMY NEW ENGLAND DIVISION CORPS OF ENGINEERS WALTHAM, MASS.	
DR. BY W. F.	WATER RESOURCES DEVELOPMENT PROJECT PARK RIVER LOCAL PROTECTION AUXILIARY CONDUIT PLAN AND GEOLOGIC LOG PROFILE STA. 73+50 TO STA. 67+00 HARTFORD CONNECTICUT
	APPROVED DATE
	CHIEF, ENGINEERING DIVISION
	SCALE / SPEC. NO. / DRAWING NUMBER

CORPS OF ENGINEERS U. S. ARMY



PLAN
SCALE 1"=20'



GEOLOGIC LOG PROFILE
SCALE 1"= 20'

GRAPHIC SCALES
1" = 20' 20' 0 20' 40'

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

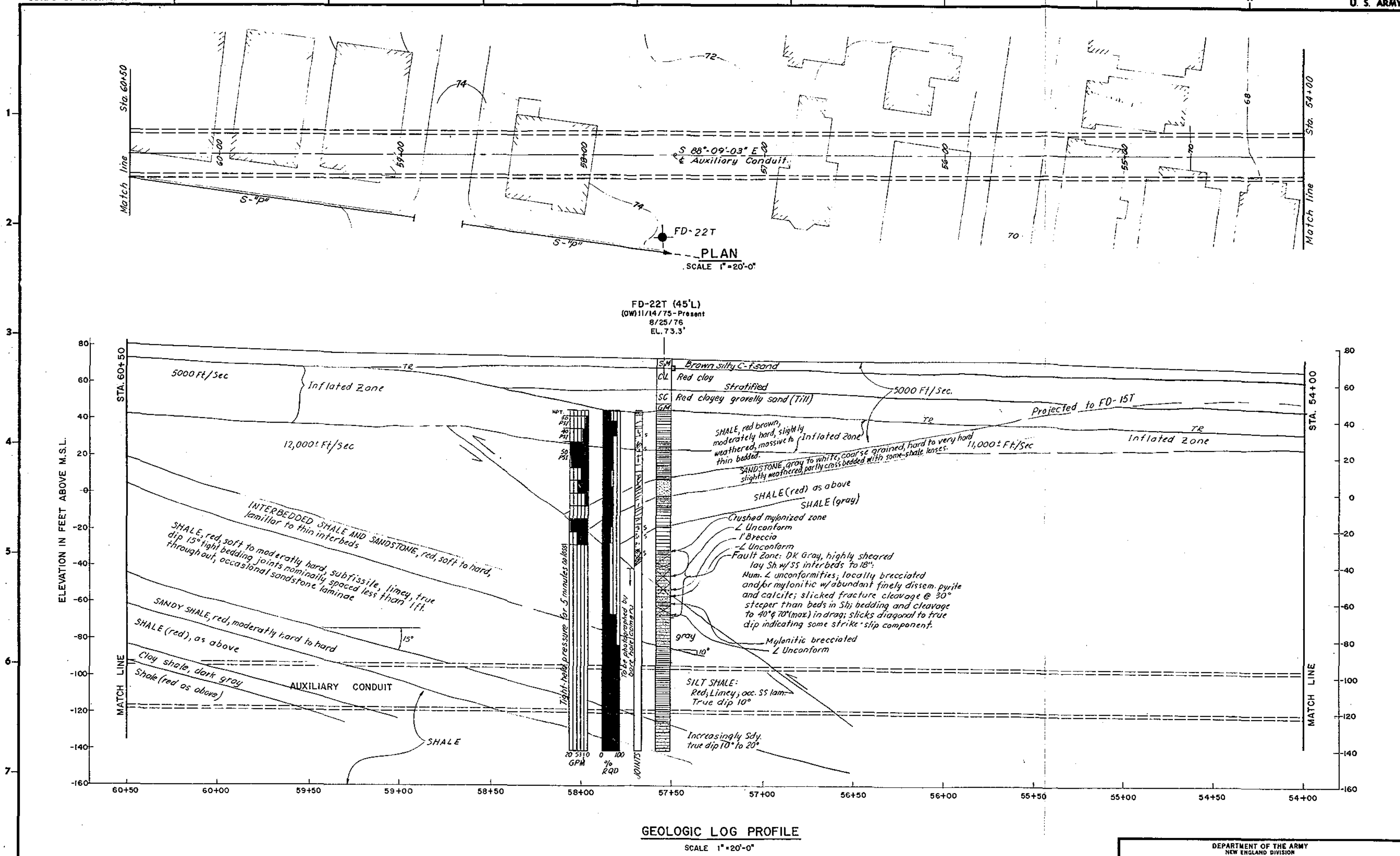
DES. BY | DR. BY W.F. | CK. BY

WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
AUXILIARY CONDUIT
PLAN AND GEOLOGIC LOG PROFILE
STA. 67+00 TO STA. 60+50
HARTFORD CONNECTICUT

SUBMITTED: | CHIEF, SECTION | APPROVAL RECOMMENDED: | CHIEF, TECH. ENG. BRANCH | REVIEWED: | PROJECT ENGINEER | APPROVAL RECOMMENDED: | CHIEF, BRANCH
APPROVED | DATE
CHIEF, ENGINEERING DIVISION
SCALE | SPEC. NO.
DRAWING NUMBER
SHEET

PLATE 9A-10

CORPS OF ENGINEERS — U. S. ARMY



DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

DR. BY W. F.

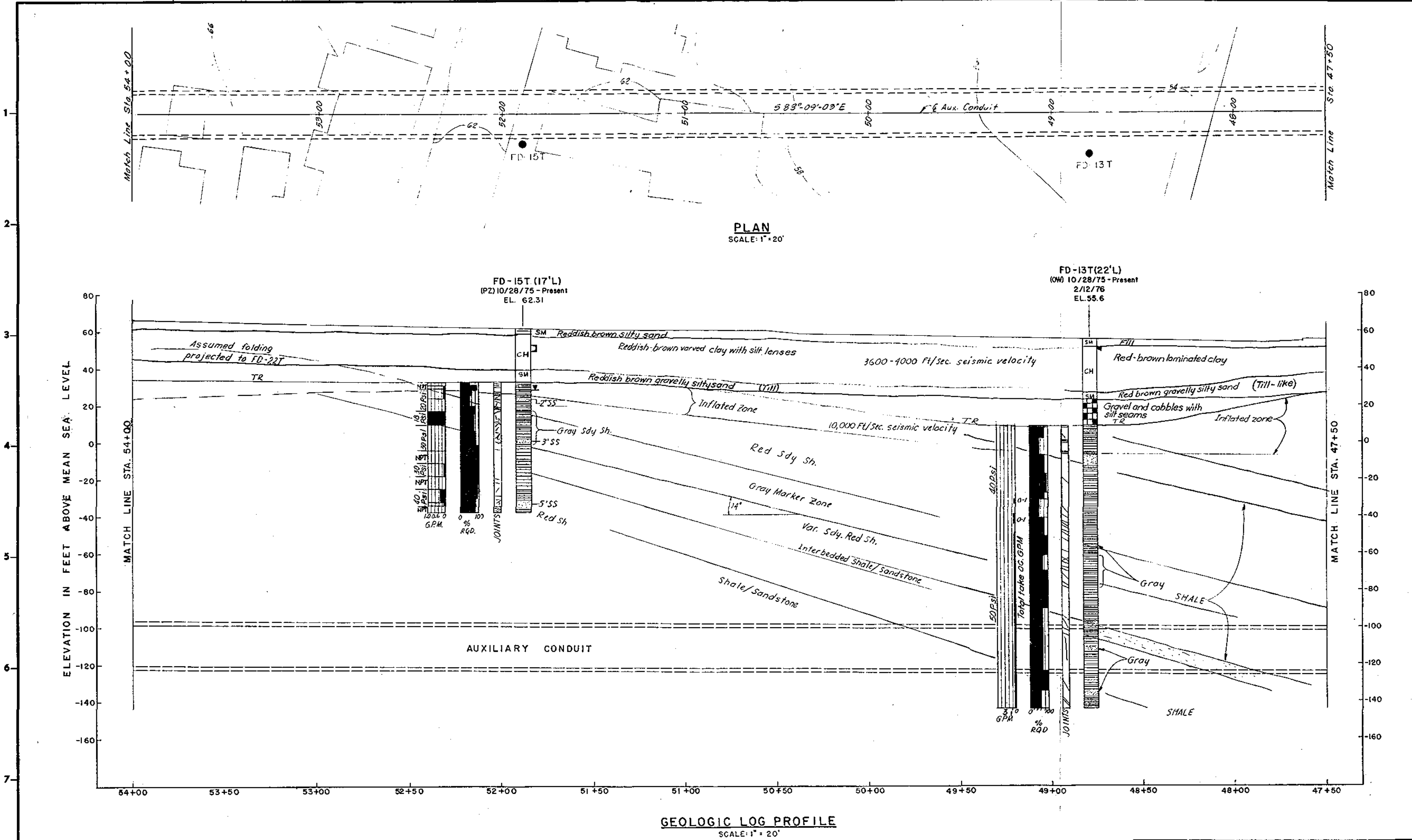
WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
AUXILIARY CONDUIT
PLAN AND GEOLOGIC LOG PROFILE
STA. 60+50 TO STA. 54+00
HARTFORD — CONNECTICUT

APPROVED — DATE

SCALE — SPEC. NO. — DRAWING NUMBER — SHEET

PLATE 9A-11

CORPS OF ENGINEERS U. S. ARMY



GRAPHIC SCALE
1" = 20'

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

DES. BY | DR. BY J.O. | CK. BY

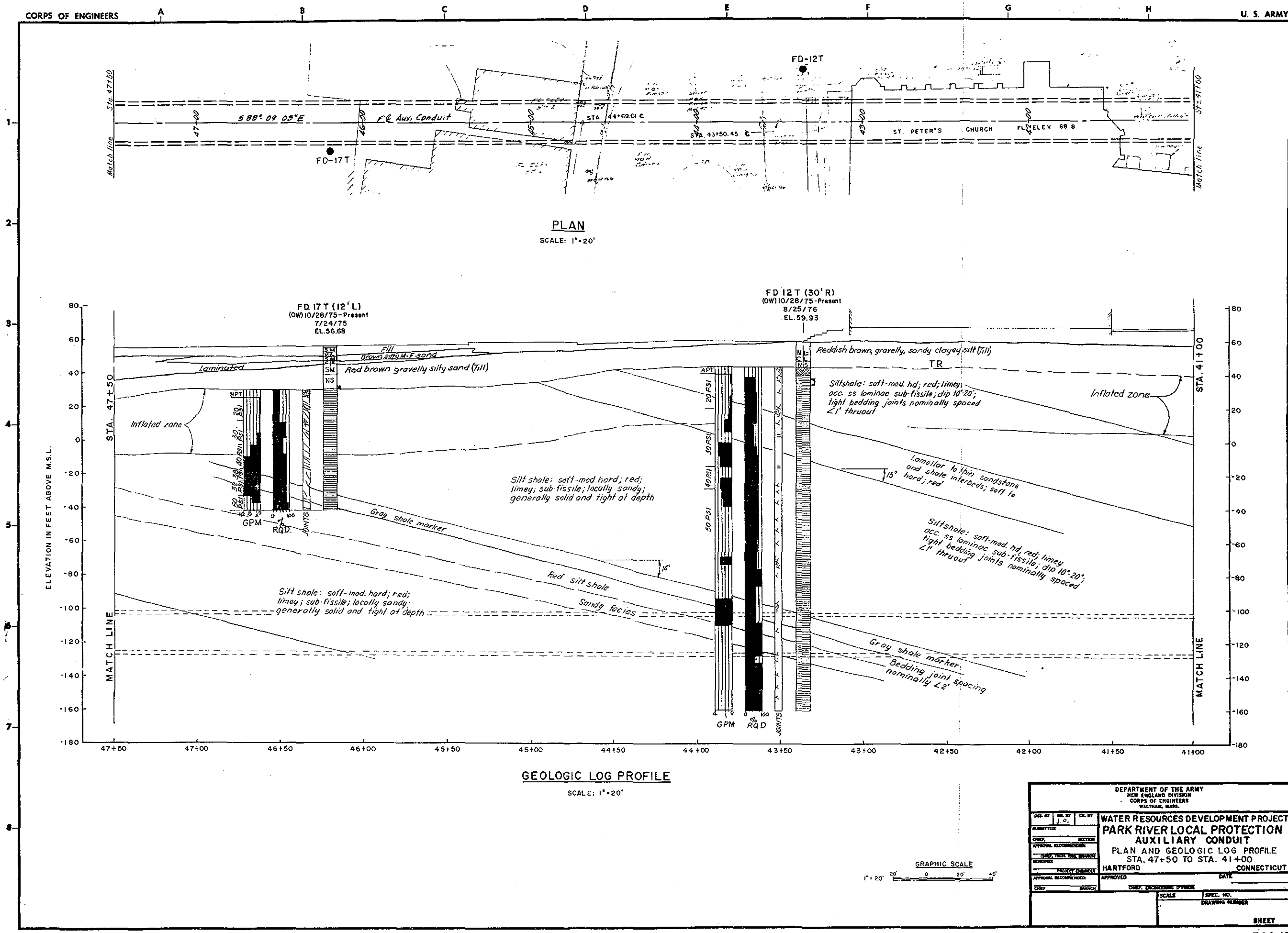
SUBMITTED: CHIEF, SECTION
APPROVAL RECOMMENDED: CHIEF, TECH. ENG. BRANCH
REVIEWED: PROJECT ENGINEER
APPROVAL RECOMMENDED: CHIEF BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
AUXILIARY CONDUIT
PLAN AND GEOLOGIC LOG PROFILE
STA. 54+00 TO STA. 47+50
HARTFORD CONNECTICUT

APPROVED DATE
CHIEF, ENGINEERING DIVISION

SCALE | SPEC. NO.
DRAWING NUMBER
SHEET

PLATE 9A-12



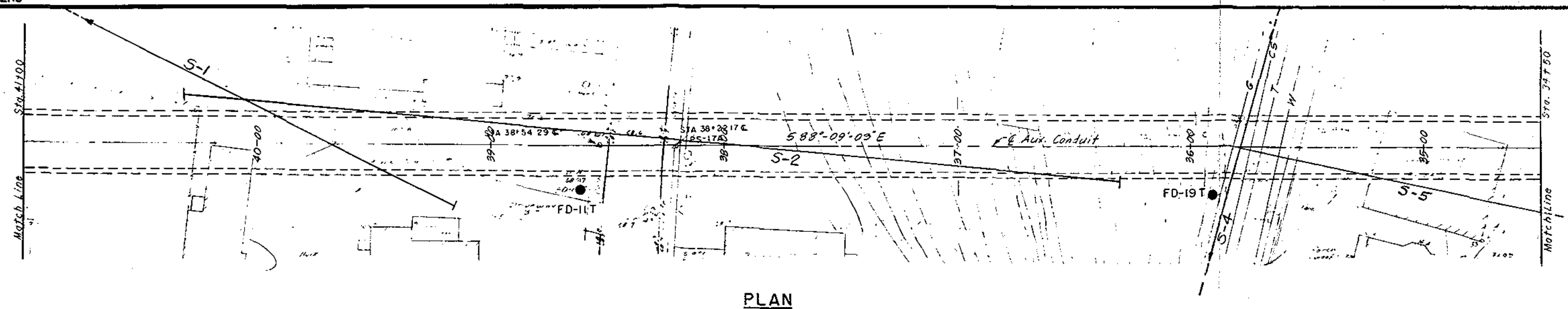
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
AUXILIARY CONDUIT
PLAN AND GEOLOGIC LOG PROFILE
STA. 47+50 TO STA. 41+00
HARTFORD CONNECTICUT

PLATE 9A-13

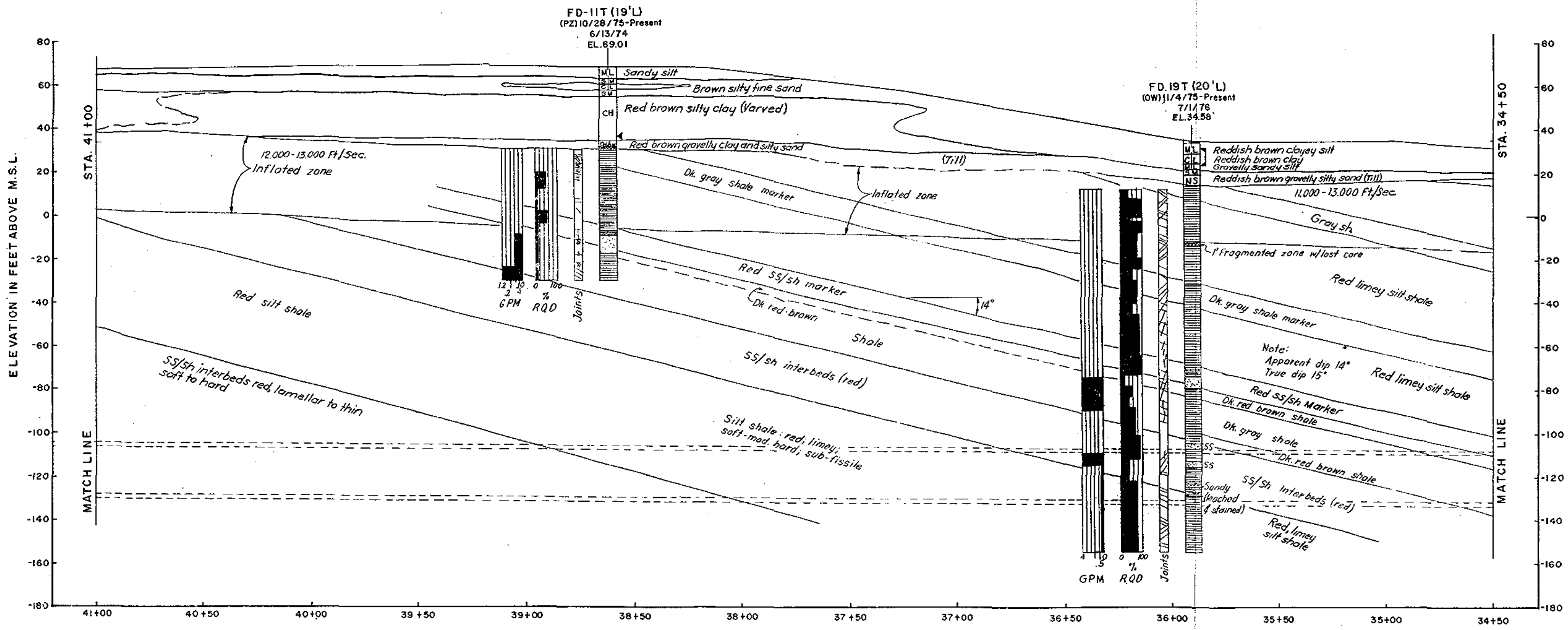
CORPS OF ENGINEERS

U. S. ARMY



PLAN

SCALE: 1" = 20'



GEOLOGIC LOG PROFILE

SCALE: 1" = 20'

GRAPHIC SCALE 1" = 20' 20' 0 20' 40'

GRAPHIC SCALE 1" = 20' 20' 0 20' 40'

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
AUXILIARY CONDUIT
PLAN AND GEOLOGIC LOG PROFILE
STA. 41+00 TO STA. 34+50
HARTFORD CONNECTICUT

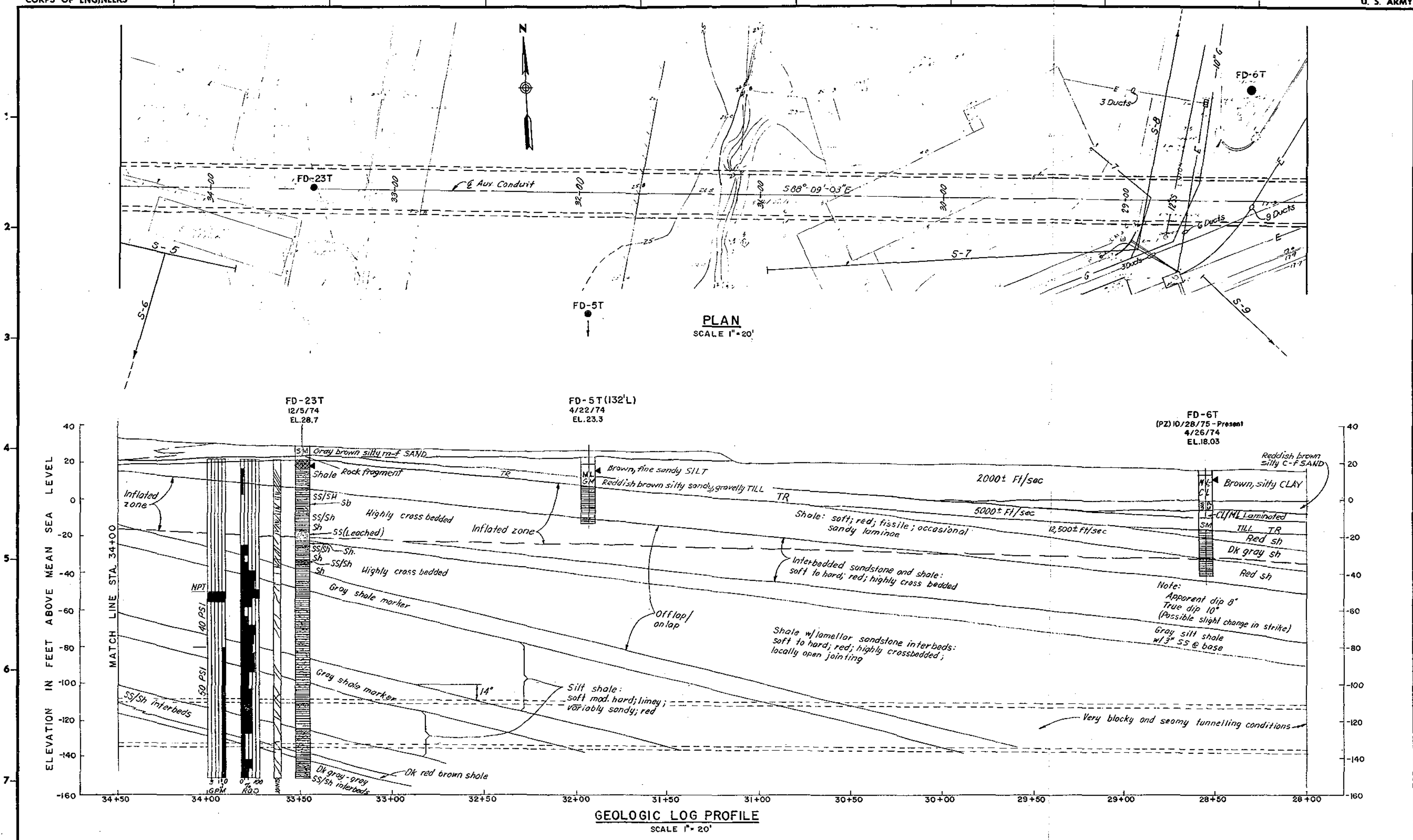
APPROVED DATE

CHIEF, ENGINEERING DIVISION

SCALE | SPEC. NO. | DRAWING NUMBER | SHEET

PLATE 9A-14

CORPS OF ENGINEERS U. S. ARMY



GRAPHIC SCALE 1"=20' 20' 0 20' 40'

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

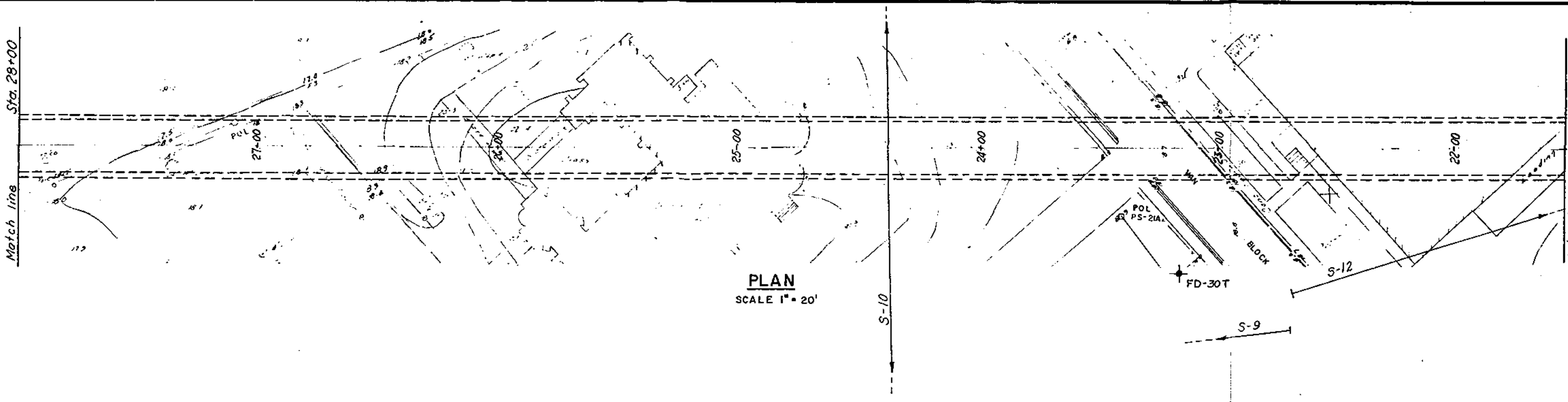
WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
AUXILIARY CONDUIT
PLAN AND GEOLOGIC LOG PROFILE
STA. 34+50 TO STA. 28+00
HARTFORD CONNECTICUT

DES. BY	DR. BY	CK. BY	APPROVED	DATE
SUBMITTED:				
CHIEF, SECTION			CHIEF, ENGINEERING DIVISION	
APPROVAL RECOMMENDED:			SCALE	SPEC. NO.
CHIEF, TECH. ENG. BRANCH				DRAWING NUMBER
REVIEWED: PROJECT ENGINEER				
APPROVAL RECOMMENDED: CHIEF BRANCH				SHEET

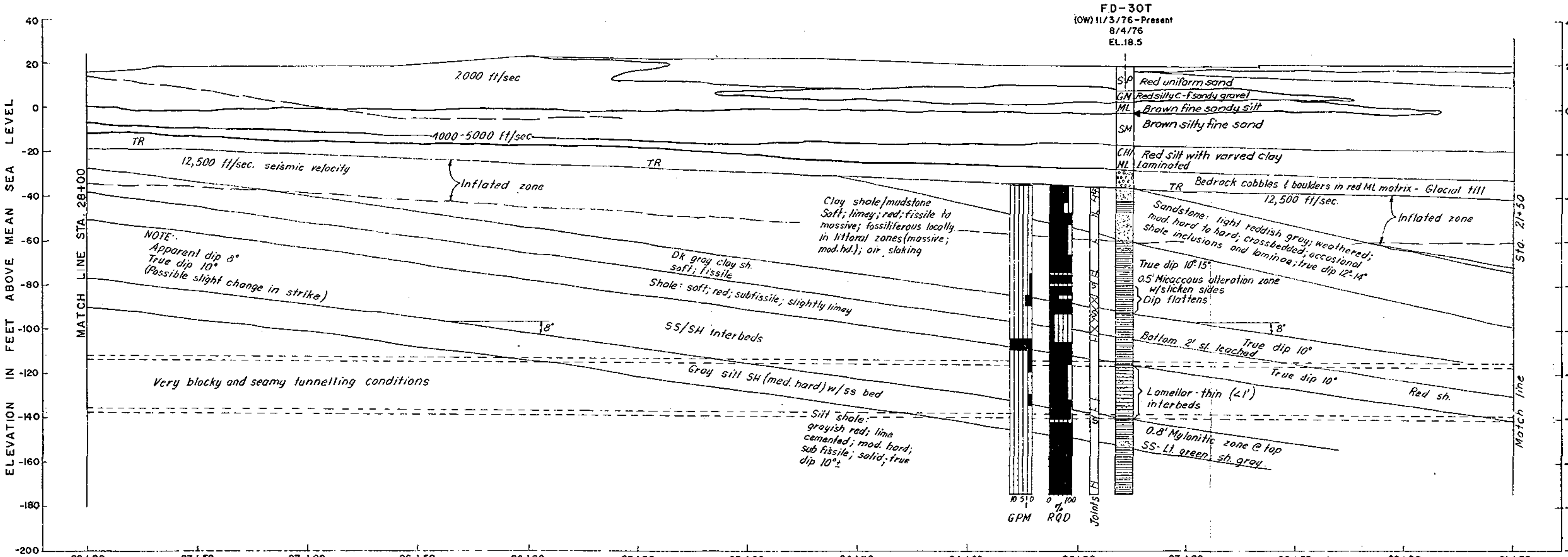
PLATE 9A-15

CORPS OF ENGINEERS

U. S. ARMY



PLAN
SCALE 1" = 20'



GEOLOGIC LOG PROFILE
SCALE 1" = 20'

GRAPHIC SCALE
1" = 20'

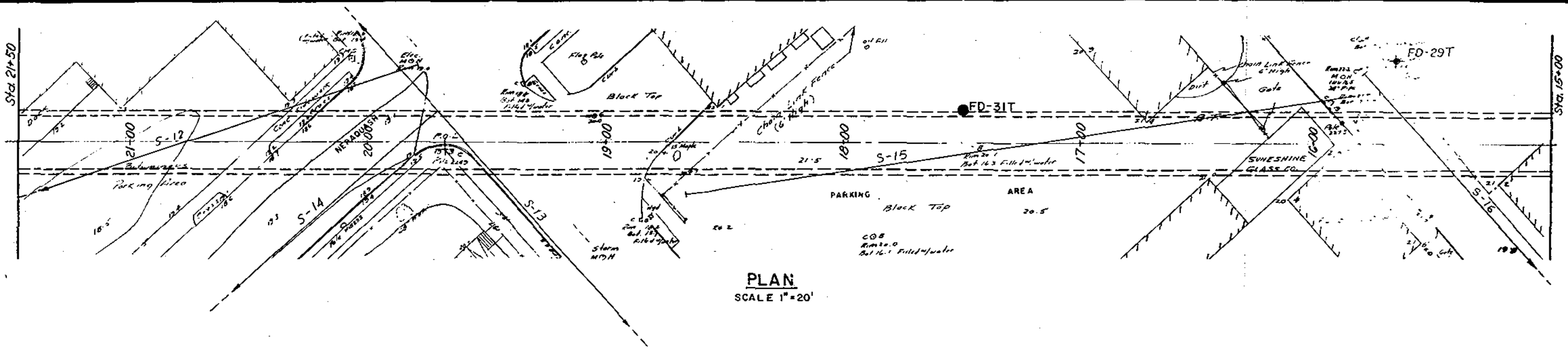
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
AUXILIARY CONDUIT
PLAN AND GEOLOGIC LOG PROFILE
STA. 28+00 TO STA. 21+50
HARTFORD CONNECTICUT

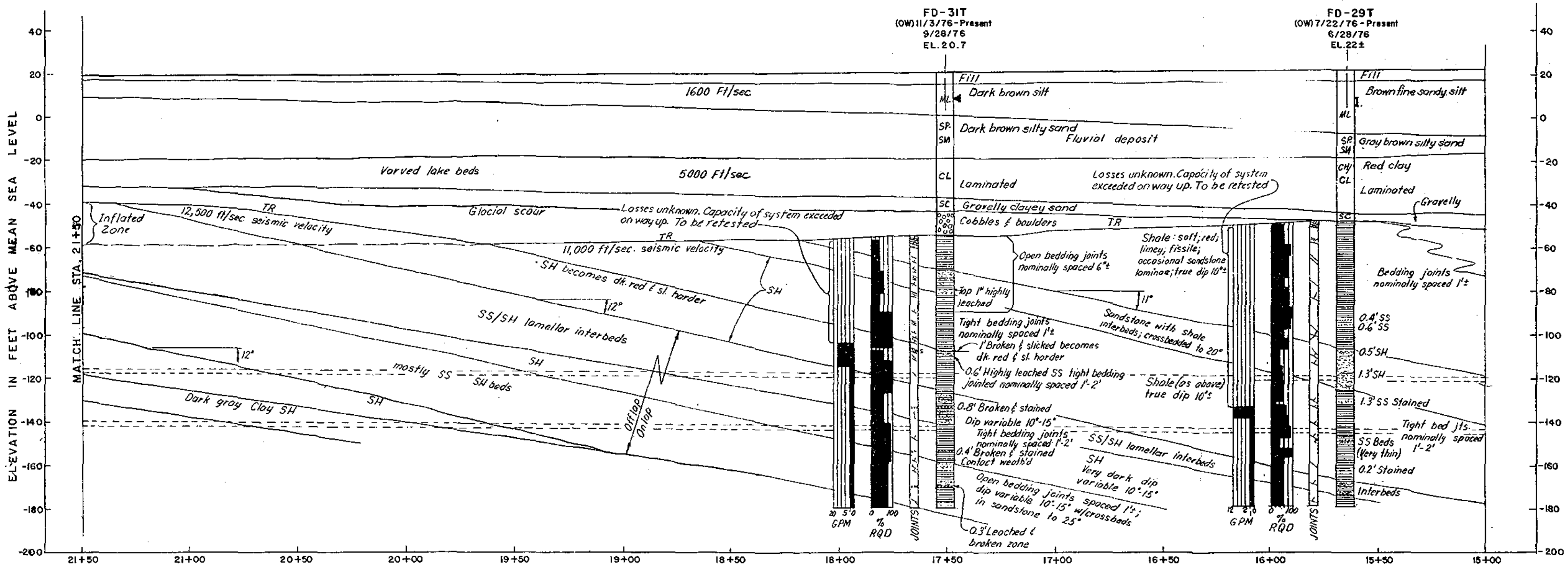
APPROVED DATE

PLATE 9A-16

CORPS OF ENGINEERS | U. S. ARMY



PLAN
SCALE 1"=20'



GEOLOGIC LOG PROFILE
SCALE 1"= 20'

GRAPHIC SCALE
1"= 20' 20' 0 20' 40'

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
AUXILIARY CONDUIT
PLAN AND GEOLOGIC LOG PROFILE
STA. 21+50 TO STA. 15+00
HARTFORD CONNECTICUT

PLATE 9A-17

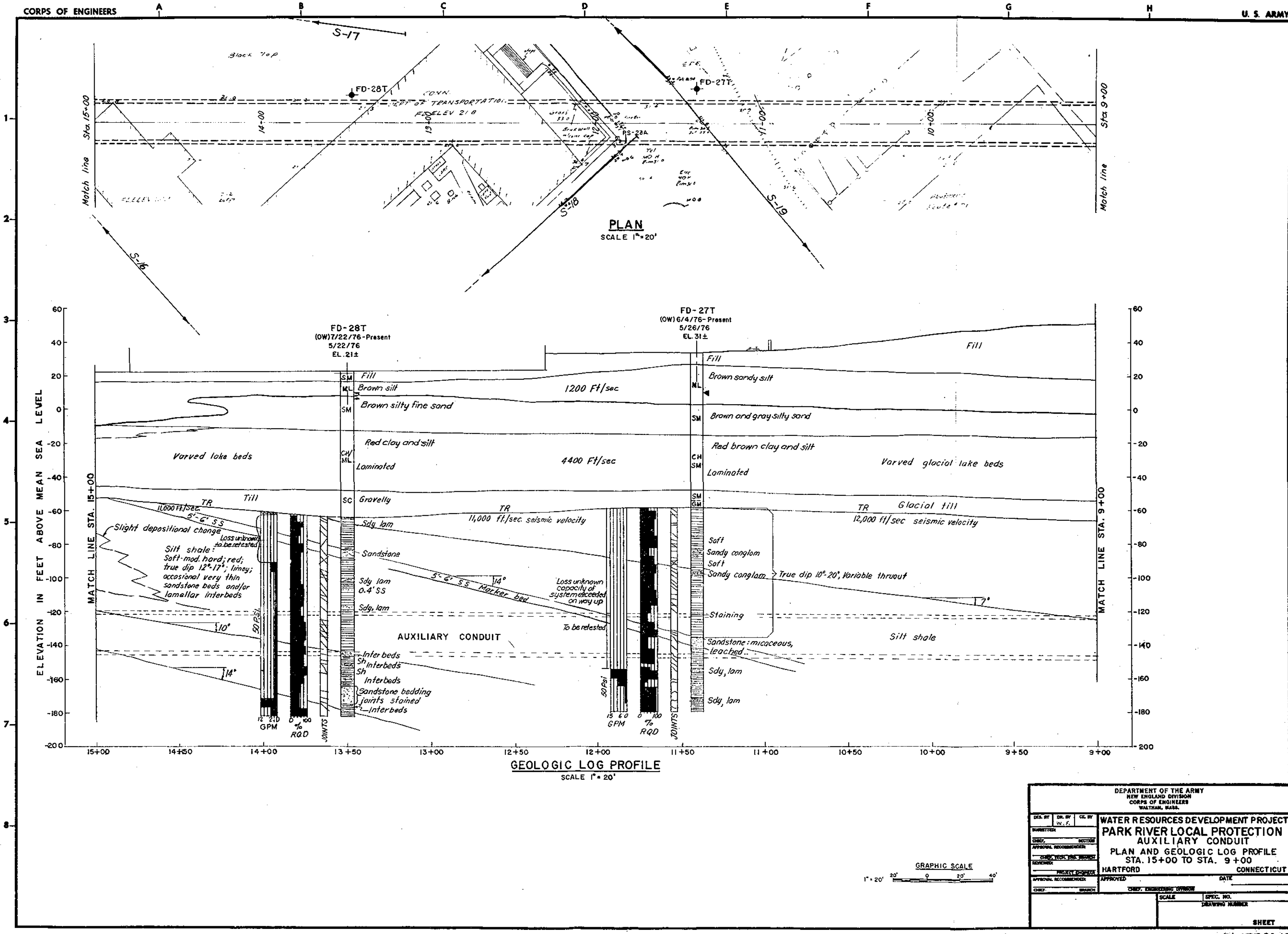
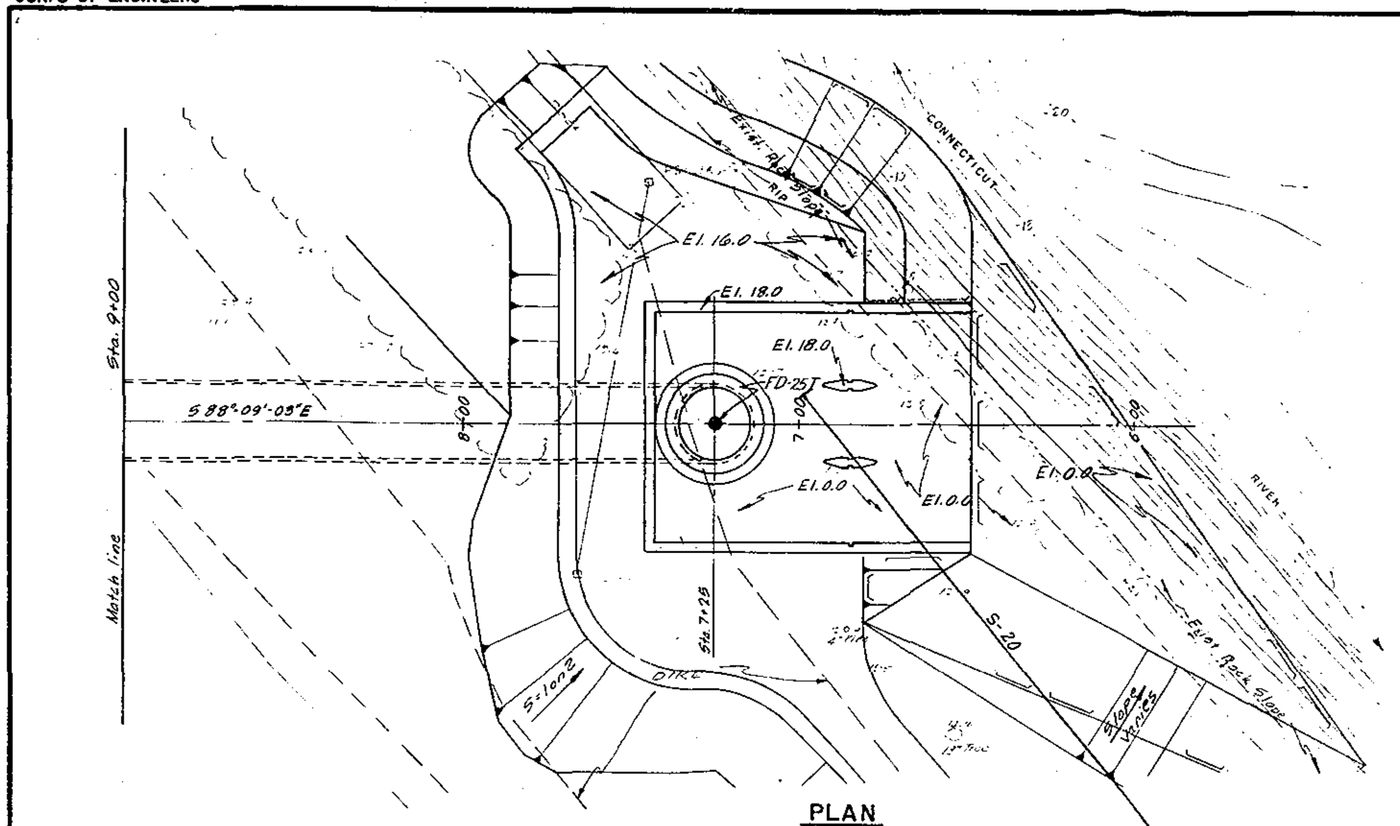
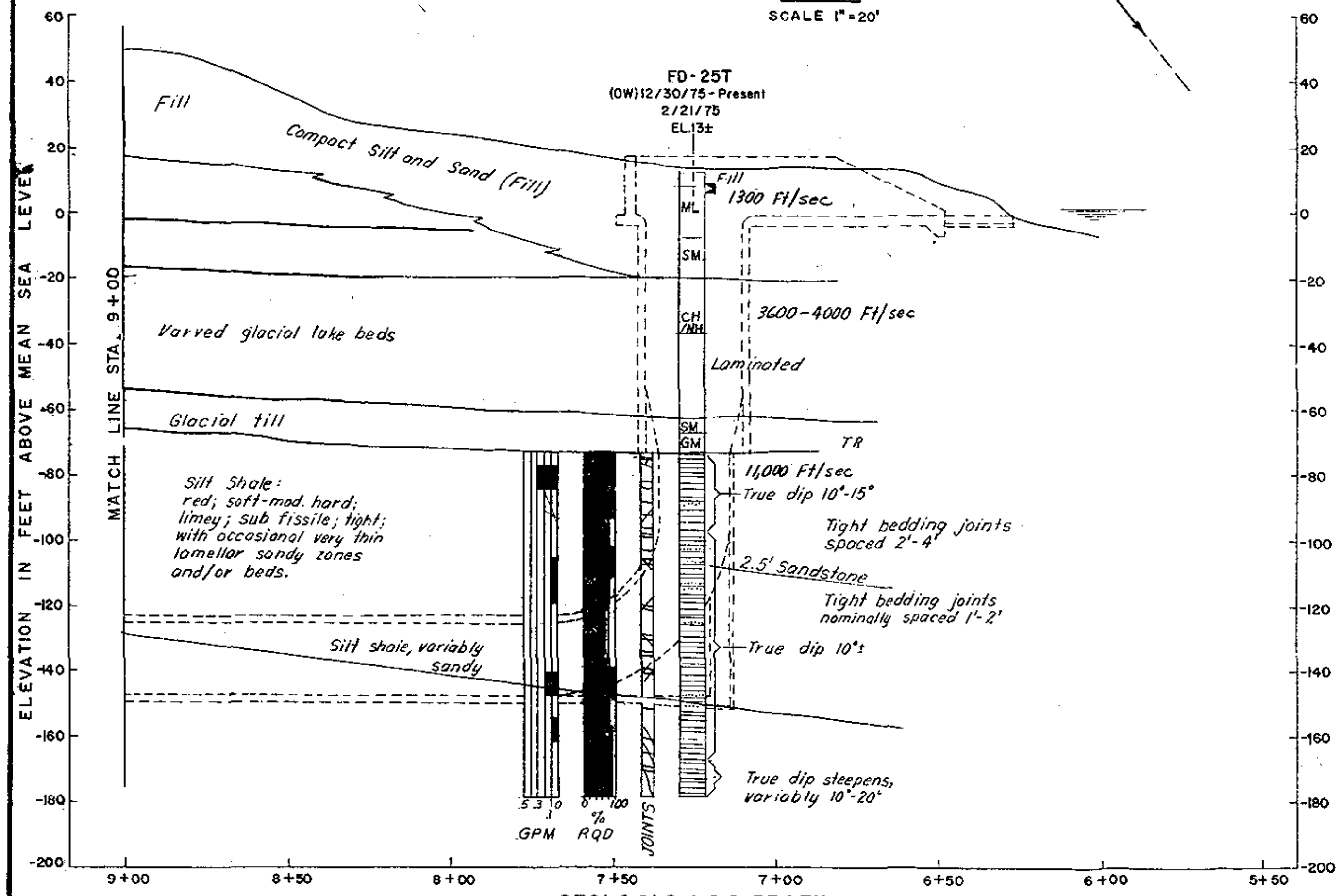

CORPS OF ENGINEERS
U. S. ARMY
PLAN
SCALE 1" = 20'
FD-28T
FD-27T
S-16
S-17
S-18
S-19
PS-28A
Block Top
CONN. DEPT OF TRANSPORTATION
Sta. 15+00
Match line
Sta. 9+00
Match line
14+00
13+00
12+00
11+00
10+00
GEOLOGIC LOG PROFILE
SCALE 1" = 20'
ELEVATION IN FEET ABOVE MEAN SEA LEVEL
MATCH LINE STA. 15+00
MATCH LINE STA. 9+00
FD-28T
(OW) 7/22/76 - Present
5/22/76
EL. 21±
FD-27T
(OW) 6/4/76 - Present
5/26/76
EL. 31±
Fill
Brown silt
Brown silty fine sand
Red clay and silt
Laminated
Gravelly
Sdy lam
Sandstone
Sdy lam
0.4' SS
Sdy. lam
Inter beds
Sh Interbeds
Sh Interbeds
Sandstone bedding joints stained
Interbeds
Brown sandy silt
Brown and gray silty sand
Red brown clay and silt
Laminated
Soft
Sandy conglom
Soft
Sandy conglom
True dip 10°-20°, Variable thruout
Staining
Sandstone: micaceous, leached
Sdy, lam
Sdy, lam
1200 Ft/sec
4400 Ft/sec
Varved lake beds
Varved glacial lake beds
Till
TR
11,000 Ft/sec
5'-6' SS
TR
11,000 ft./sec. seismic velocity
TR
Glacial till
12,000 ft/sec seismic velocity
Slight depositional change
Loss unknown to be retested
Silt shale:
Soft-mod. hard; red; true dip 12°-17°; limey; occasional very thin sandstone beds and/or lamellar interbeds
5'-6' SS Marker bed
14°
Loss unknown capacity of system exceeded on way up
To be retested
7°
10°
14°
AUXILIARY CONDUIT
Silt shale
50 PSI
50 PSI
GPM
% RQD
JOINTS
GPM
% RQD
JOINTS
15+00
14+50
14+00
13+50
13+00
12+50
12+00
11+50
11+00
10+50
10+00
9+50
9+00
GRAPHIC SCALE
1" = 20'
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.
WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
AUXILIARY CONDUIT
PLAN AND GEOLOGIC LOG PROFILE
STA. 15+00 TO STA. 9+00
HARTFORD
CONNECTICUT
APPROVED
DATE
SCALE
SPEC. NO.
DRAWING NUMBER
SHEET

PLATE 9A-18



PLAN
SCALE 1" = 20'

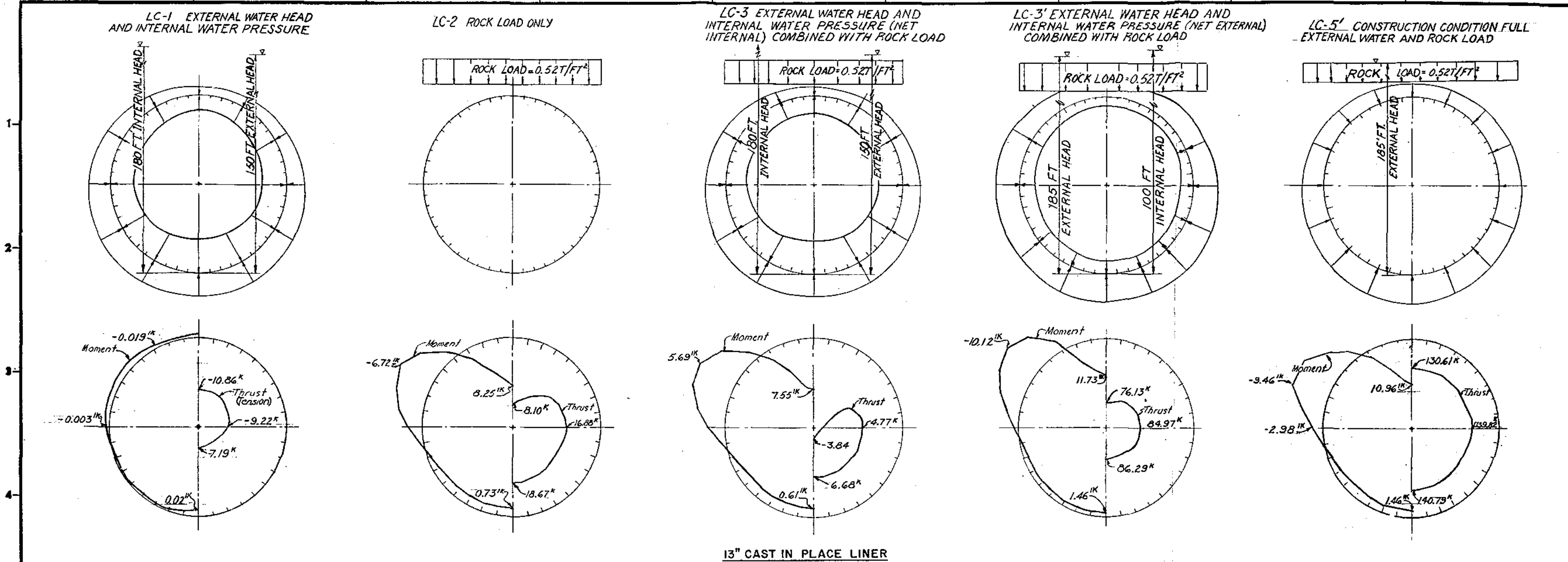


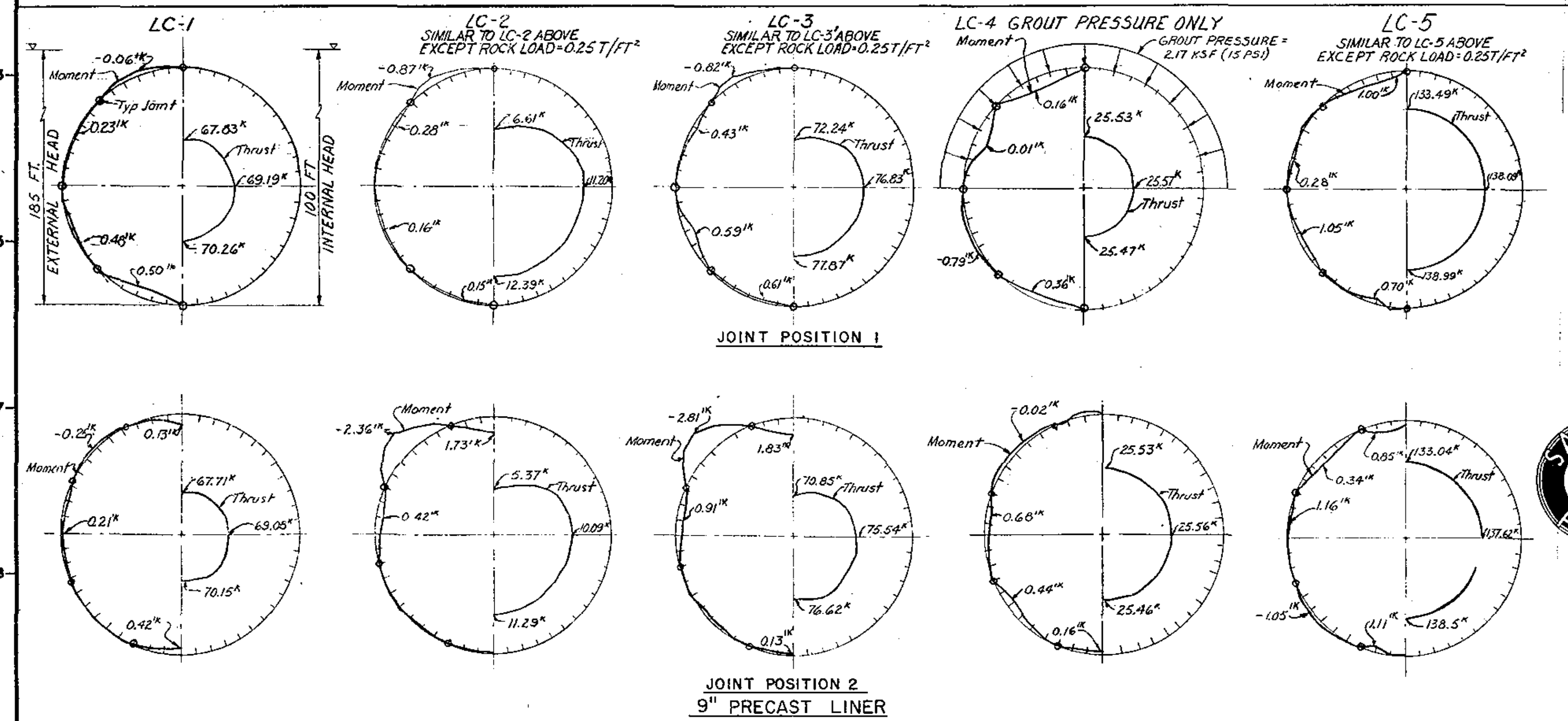
GEOLOGIC LOG PROFILE
SCALE 1" = 20'

GRAPHIC SCALE
1" = 20' 20' 0 20' 40'

DEPARTMENT OF THE ARMY NEW ENGLAND DIVISION CORPS OF ENGINEERS WALTHAM, MASS.	
WATER RESOURCES DEVELOPMENT PROJECT PARK RIVER LOCAL PROTECTION AUXILIARY CONDUIT PLAN AND GEOLOGIC LOG PROFILE STA. 9+00 TO STA. 6+00	
HARTFORD	CONNECTICUT
APPROVED	DATE
SCALE	SPEC. NO.
DRAWING NUMBER	SHEET

CORPS OF ENGINEERS — U. S. ARMY





NOTE:
1. All loading cases shown on this sheet are for Tunnel sections which would require nominal rock bolt support.
2. LC-4 is loaded spring line to spring line.

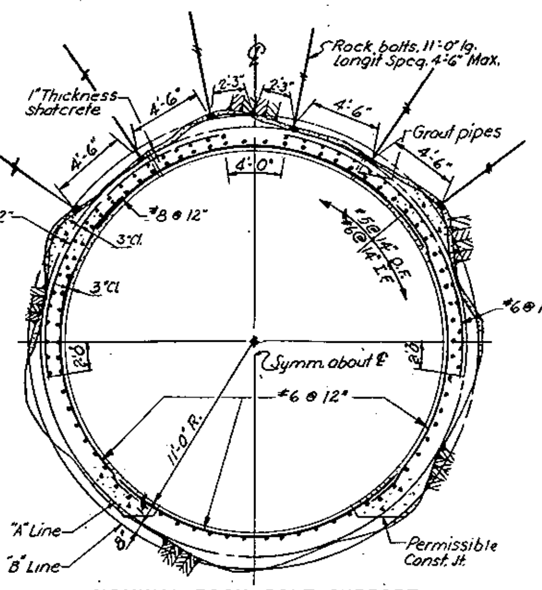
REVISION	DATE	DESCRIPTION	BY

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

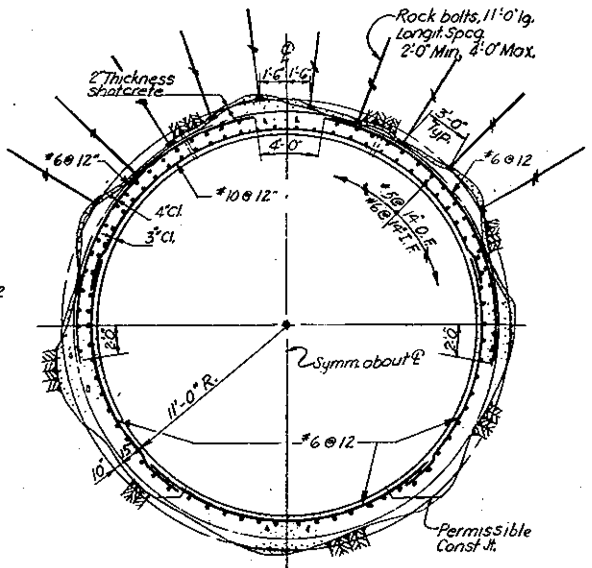
WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
AUXILIARY CONDUIT
TUNNEL LINING
LOAD MOMENT AND THRUST DIAGRAMS
HARTFORD — CONNECTICUT

APPROVED — DATE
SCALE — SPEC. NO. — DRAWING NUMBER
SHEET

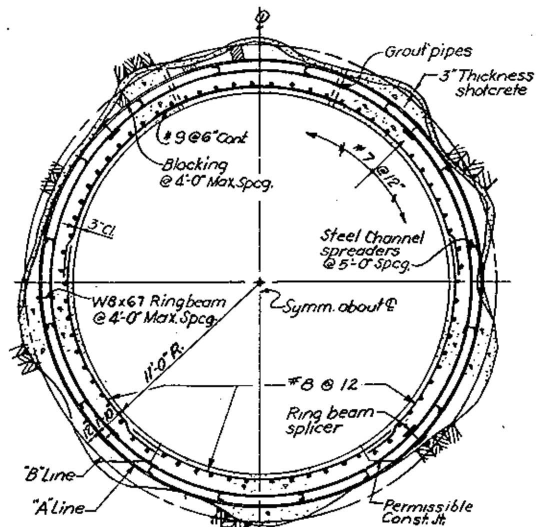
PLATE 9A-20



NOMINAL ROCK BOLT SUPPORT



HEAVY ROCK BOLT SUPPORT

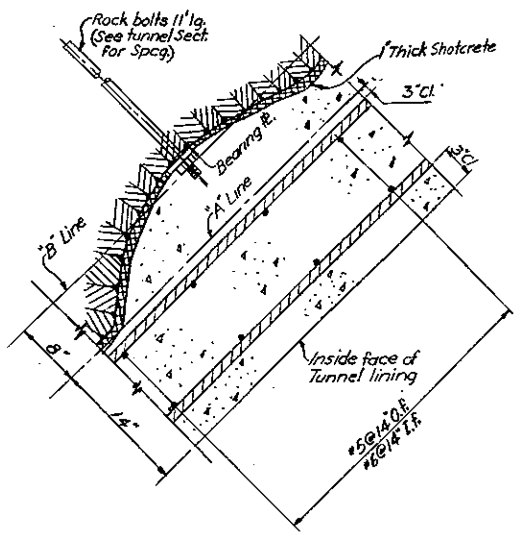


STRUCTURAL STEEL SUPPORTED

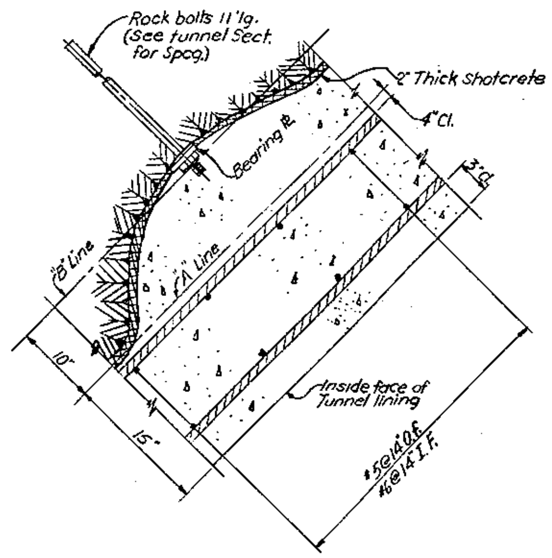
TRANSVERSE TUNNEL SECTIONS

(DRILL AND BLAST EXCAVATION)

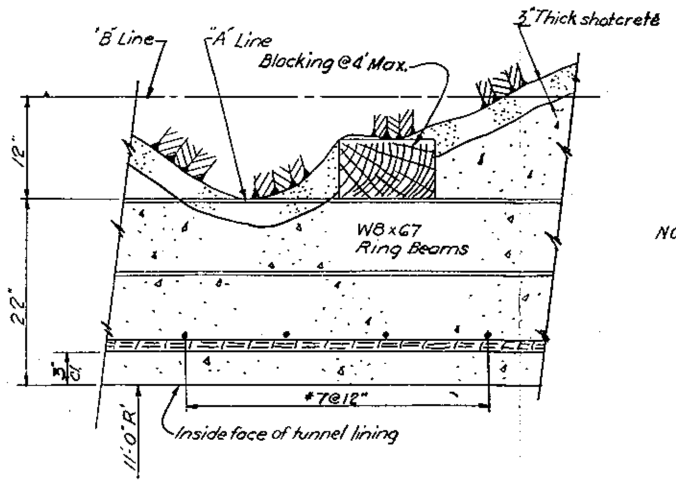
SCALE 1/4" = 1'-0"



NOMINAL ROCK BOLT SUPPORT



HEAVY ROCK BOLT SUPPORT



STRUCTURAL STEEL SUPPORTED

NOTES:
1. The tunnel lining will be placed in 30 foot long monoliths.
2. Longitudinal reinforcement will be continuous thru monolith joints.
3. All monolith joints will have waterstops.

TUNNEL SUPPORT AND LINER DETAILS

(DRILL AND BLAST EXCAVATION)

SCALE 1 1/2" = 1'-0"

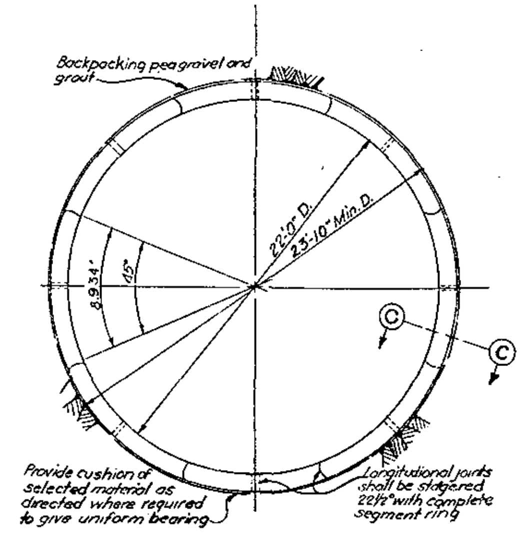
SAFETY PAYS

GRAPHIC SCALES
1/4" = 1'-0"
1 1/2" = 1'-0"

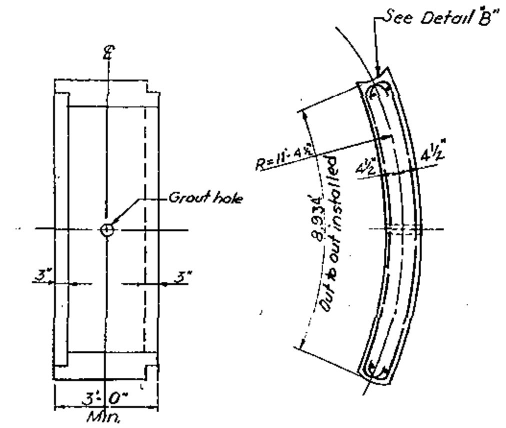
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
AUXILIARY CONDUIT
DETAILS-CAST-IN-PLACE LINER
DRILL AND BLAST EXCAVATION
HARTFORD CONNECTICUT

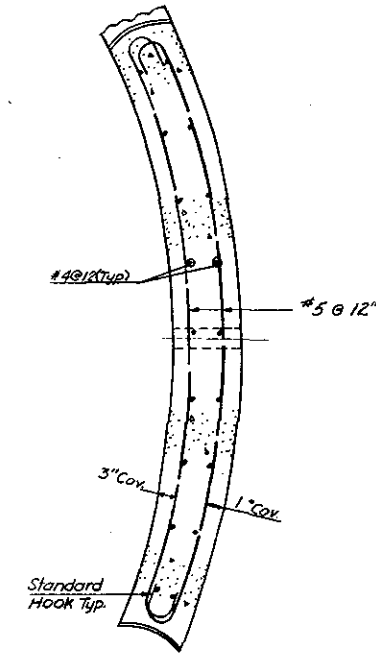
CORPS OF ENGINEERS — U. S. ARMY



TRANSVERSE TUNNEL SECTION
PRECAST LINER ALTERNATE
SCALE 1/4" = 1'-0"

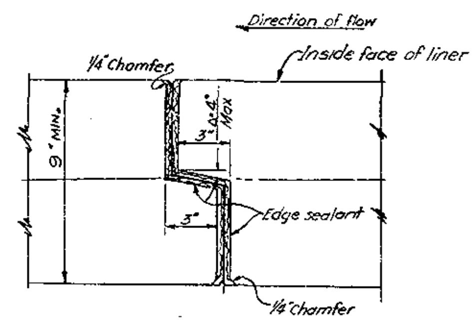


DETAIL
PRECAST LINER SEGMENT
SCALE 1/2" = 1'-0"

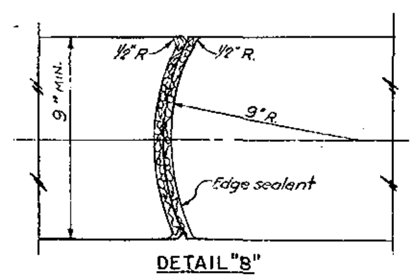


REINFORCEMENT DETAILS
PRECAST LINER
SCALE 1" = 1'-0"

NOTES:
1. Design of precast segments based on a compressive strength of 5000 pounds per square inch at 28 days.
2. The grouting procedure will have to be carefully monitored during the construction phase to insure uniform pressures throughout the cross section.



SECTION C-C
TRANSVERSE JOINT DETAIL
SCALE 3" = 1'-0"



DETAIL "B"
LONGITUDINAL JOINT DETAIL
SCALE 3" = 1'-0"

SAFETY PAYS

GRAPHIC SCALE
1" = 20' 20' 0 20' 40'

REVISION	DATE	DESCRIPTION	BY

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
AUXILIARY CONDUIT
DETAILS-PRECAST LINER

HARTFORD — CONNECTICUT

APPROVED — DATE

CHIEF, ENGINEERING DIVISION

SCALE — SPEC. NO. — DRAWING NUMBER — SHEET

APPENDIX A

TYPICAL DRILL LOG AND ROCK CORE

CONTENTS

Site:	Boring No.	Page ____ of ____

DEPTH		CORE/SAMPLE			BLOWS PER FT. / CORE RECV'Y	SAMPLING AND CORING OPERATIONS					CLASSIFICATION OF MATERIALS
	Ft.	NO.	SIZE	DEPTH RANGE							
ELEV			TEST		%	RQD %	PT GPM	JOINTS ∠	JOINTS CODE	JOINTS REMARKS	

Laboratory Test Sample No.

Per cent Core Recovery Per Run

ROCK SYMBOL

SHALE

INTERBEDDED SHALE AND SANDSTONE

SANDSTONE

LIMY ZONES

Joint and Dip Angle

Pressure test takes in GPM at 50 p.s.i.

Rock Quality Designation - Per cent of core in pieces in excess of 4 inches in length (per run)

JOINT CODE KEY

3. JOINT ROUGHNESS NUMBER

   A. Discontinuous
   B. Rough or irregular, undulating
   C. Smooth, undulating
   D. Slickensided, undulating
   E. Rough or irregular, planar
   F. Smooth, planar
   G. Slickensided, planar

4. JOINT ALTERATION NUMBER

   A. Tightly healed, hard, non-softening, impermeable filling
   B. Unaltered joint walls, surface staining only
   C. Slightly altered joint walls
   D. Silty or sandy-clay coatings, small clay fraction (non-softening)
   E. Softening or low friction clay mineral coatings (discontinuous coatings, 1-2mm or less in thickness)

LEGEND FOR TYPICAL BORING LOG

Site: Park River Tunnel	Boring No. FD-30T	Page 1 of 9

DEPTH 1" = 2'	CORE/SAMPLE NO.	SIZE	DEPTH RANGE	BLOWS PER FT. / CORE RECVY TR	RQD	Joints ∠	#	Remarks	Symb.	CLASSIFICATION OF MATERIALS
54					0					CLAY SHALE: red; broken; soft
56				100	90		Str 3e 4c Bd 3e 4e	Bedding Jts w/Strike Jts normal; Sh bed		SANDSTONE: mod. hd-hd; lt red gray; loc. calc.; occ. sh incls & lam.; dip 12°-14°; micaceous; sl. leached;
58				100			3a 4b			(2-Incipient str jts below 58')
60										
62				100	60					CLAY SHALE: red; sub-fissile; slakes readily when wet; dip 10°-15°; no joints; soft
64										
66										SANDSTONE: as above
68				100	10		Dp 3b 4d Str 3e 4c Bd 3e 4e	Near vert. irreg. dip jt intersecting normal strike & bedding set		
70										

Site:	Boring No.	Page 2 of 9
Park River Tunnel	FD-30T	

Depth (1" = 2')	Core/Sample No.	Size	Depth Range	Blows per ft. / Core Recv'y	RQD	Joints ∡	Joints #	Joints Remarks	Symb.	Classification of Material
70					100		3e 4c / 3e 4e	Strike joint normal to: Bedding Jt		SANDSTONE: (Cont'd)
72				98						Inter beds @ 10°-15° very calc.
74					85					Num. Shaley stringers increasing w/ depth; dip 12°-14°
76										
78				100			3e 4c / 3f 4e	Strike joint normal to: Bedding Jt		Depth Primary Weath.
										CLAY SHALE: soft; red; sub-fissile slakes readily when wet; dip <10°; occ. tiny calcite stringers ∥ bedding; Top 0.4' very calc. littoral zone; no joints
80										
82				100	70					Highly calc. fossilif. littoral zone w/ hd sandy interbeds to 3"; no joints
84										
86							3e 4d	Dip Jt @ 75°-80° North (Cont'd)		SHALE/MUDSTONE: (over)

D FORM 55A (Test) MAR 71

Boring No. FD-30T

Site: Park River Tunnel	Boring No. FD-30T	Page 8 of 9

DEPTH 1" = 2'	CORE/SAMPLE NO.	SIZE	DEPTH RANGE	BLOWS PER FT. / CORE RECVY	RQD	Joints ∠	#	Remarks	Symb.	CLASSIFICATION OF MATERIALS
86										SHALE/MUDSTONE (Cont'd): soft-mod hd; red; thin-bedded (<1') w/ intip. shale bedding; highly calc. fossilif.; sl. sdy litto-ral dep.; subject to cyclic slaking; dip 10°-15°;
88				100				sharp Clay Sh Blended		
90					100					
92				100			3e 4d	Bd Jt Strike jt		Blended Contact
					0		3e 4e	Bd Jt Str Jt		CLAY SHALE: soft; red; sub-fissile; slakes readily when wet; dip 10°-15°; core broken due to joint;
94										
96					100		3e 4c	96.1 Shear Joint striking NW w/ 80°+dip to NE 97.3	96	Blended Contact SHALE/MUDSTONE as above
98				100						97.8 highly fossilif. zone
					0		3g 4c	Bedding Slip		98.4 Micaceous alteration zone w/ slicks on bedding; 98.9 0.1' Rec., 0.4' loss;
100					100					
102				98						

NED FORM 58A(Test) MAR 71

Boring No. FD-30T

Site: Park River Tunnel	Boring No. FD-30T	Page 4 of 9

DEPTH (1" = 2')	RQD	Joints ∠	Joints #	Remarks	CLASSIFICATION OF MATERIALS
102	100				SH /MDST (Cont'd)
			3e 4b	calcite healed random jt @ 50°E	Blended Contact Interbeds: lamellar; very limey; mod hd; dip <10°
104	43				
			3e 4b	calcite healed Shr jt @ 65° NE, Str to NW	Blended Contact SHALE: soft; red; sub-fissile; limey
106			3e 4b	Random jt @ 50°W	
108	100				
			3e 4b 3g 4e	Shr Jts @ 50° NE, Str to NW Top Calcite healed, lower w/ slicks	
110					SHALE: soft; dk gray; fissile; loc. limey; occ. calcite stringers; dip 10°±;
			111.7		
112					
			3e 4b	Series of 50° E & W dipping random jts w/ occ. near vert dip jts; some w/ calcite; core badly fragmented	
114	15				
116					
118					

Core recovery (%): 98, 100, 100, 98

FORM 55A(Test) MAR 71

Boring No. FD-30T

Site: Park River Tunnel	Boring No. FD-30 T	Page 5 of 9

DEPTH 1" = 2'	CORE/SAMPLE NO.	SIZE	DEPTH RANGE	BLOWS PER FT. / CORE RECV'Y	RQD	Joints ∡	Joints #	Joints Remarks	Symb.	CLASSIFICATION OF MATERIALS
118				98			118.6			SHALE: dk gray (cont'd)
							32 / 46	Shr Jt @ 60°NE, str NW		Bottom 2.2' sl. leached zone w/ 0.7' Lost Core
120					15					
122				84						SHALE: soft, red; sub-fissile; sl. limey thruout; dip 10°±;
							36 / 46	70°-90° str Jt		
124										1' very limey, mod. hd silty zone
126				100						
128					100					no joints
130										
132				100						
134										134.1

NED FORM 55A (Test) MAR 71

Boring No. FD-30T

Site: Park River Tunnel	Boring No. FD-30T	Page 6 of 9

DEPTH 1" = 2'	CORE/SAMPLE NO.	SIZE	DEPTH RANGE	BLOWS PER FT. / CORE RECV'Y	SAMPLING AND CORING OPERATIONS: RQD	Joints ∠	#	Remarks	Symb	CLASSIFICATION OF MATERIAL
134										134.1 SH / SS interbeds soft to hard; red to lt red-gray; variably limey & fossiliferous (littoral zones); lamellar to thin (<1'); dip 10°± to 20° where precons. slumping; no joints;
136				100						
138										
140										
142				100	80					
144										
146				100						
148										
150										SANDSTONE: hard; f.g.; limey; lt red-gry;

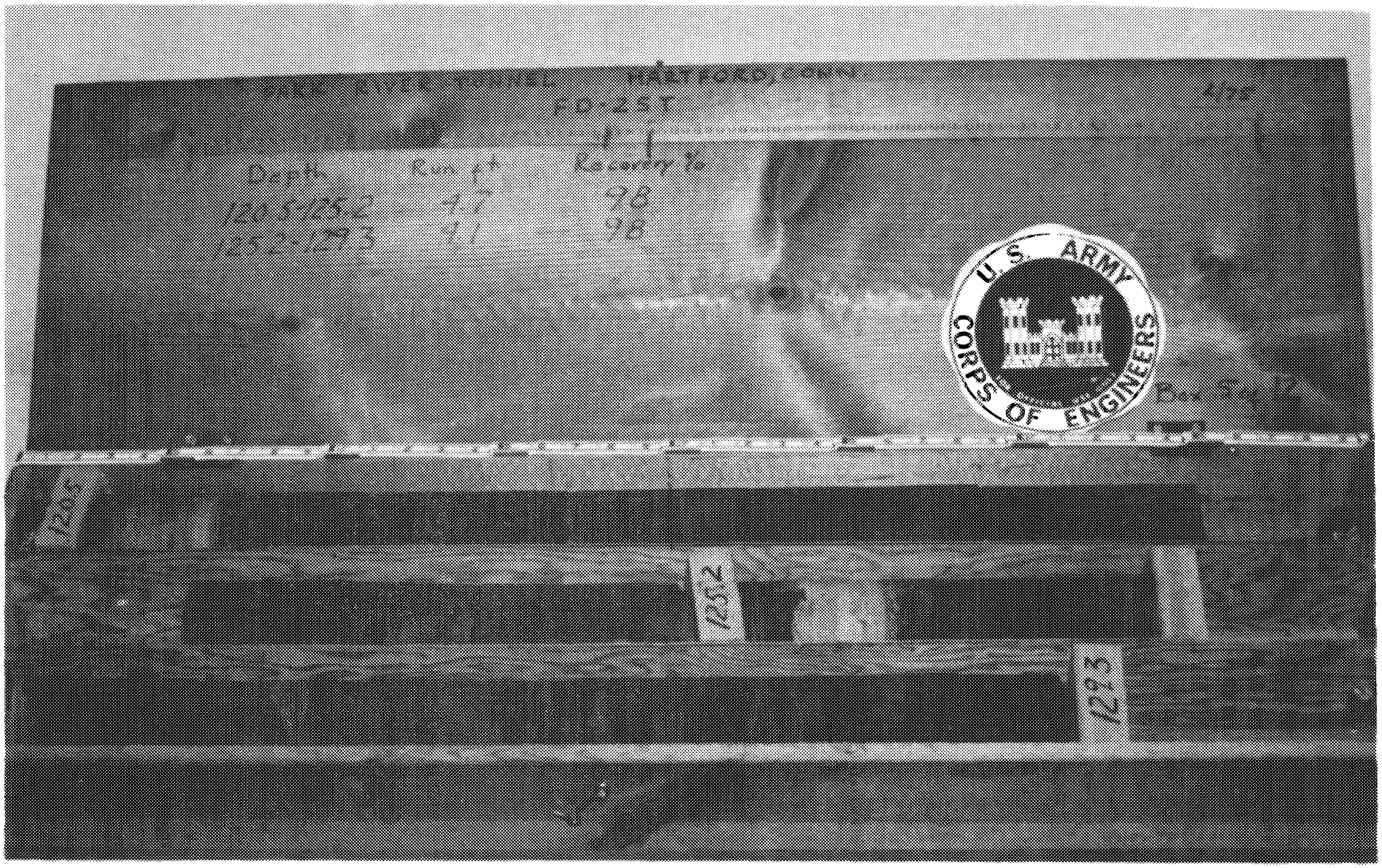
Site: Park River Tunnel	Boring No. FD-30T	Page 7 of 9

Depth (1" = 2')	Core/Sample No.	Size	Depth Range	Blows per ft. / Core Recv'y	RQD	Joints ∠	Joints #	Joints Remarks	Symb.	Classification of Materials
150					↓	/	3e 4b	Str Jt normal to beds		SS (cont'd) w/ Sh incls & stringers; thin bd
										SHALE: soft; red; calcite stringers ∥ to sub-fissility; dip 10°±; air slaking;
152				100	100					MUDSTONE: sdy; mod hd; dk red; massive; sl. limey & fossilif.
154						/	3e 4b	str Jt normal to bd w/ calcite; very sdy-limey zone		SANDY SHALE: grayish-red; mod. hd-hd; lamellar; loc. sl. limey;
156				100	90					
158				91						Blended Contact
					0		3J 4d			SILT SHALE: mod hd; dk gry; fissile; dip 10°±; solid; locally sdy & very sl. limey; no jts;
160							3g 4d	Bedding slicks w/ Calcite		159.1-159.9 = Shear Zone parallel to bedding; highly slicked & mylonitic; 0.4' lost core
162				100	100					
164										
166										

DEPTH (1" = 2')	CORE/SAMPLE NO.	SIZE	DEPTH RANGE	BLOWS PER FT. / CORE RECV'Y	RQD	Joints ∠	Joints #	Joints Remarks	Symb.	CLASSIFICATION OF MATERIALS
166										SILTSHALE (Cont'd):
168				100	100					
170										Blended Contact — SANDSTONE: f.g.; hard; lt greenish gray to 171, then lt reddish gray & sl. limey; no joints; dip 10°±
172										Blended Contact — SILT SHALE: mod. hd; lt grayish red; solid; sl. limey thruout — cemented; no jts; dip 10°±; occ. hd sdy zones thruout w/ x beds to 20°
174				100	100					
176										
178				100	100					
180										
182										

Site: Park River Tunnel	Boring No. FD-30T	Page 9 of 9

DEPTH 1"=2'	CORE/SAMPLE NO.	SIZE	DEPTH RANGE	BLOWS PER FT. / CORE RECVY	RQD	Joints ∡	#	Remarks	Symb.	CLASSIFICATION OF MATERIALS
182				100						SILT SHALE (Cont'd)
184										
186				100	100					
188							3e 4b	strike jt @ 50° very sdy zone		Increasingly sdy & harder fr. 187.4' - dip variable 10°-20°
190				100			3e 4b	strike jt @ 70°		Blended upper contact 0.4' fg Limey SS
192										
194				BB						



TYPICAL ROCK CORE

FIGURE A-3

APPENDIX B

TYPICAL BOREHOLE PHOTOGRAPHY DATA

CONTENTS

FIGURES

BORE HOLE PHOTO LOG

BORING NO.	FD-8-T
SITE	Park River Tunnel
LOCATION	Hartford, Connecticut
DATE PHOTOGRAPHED	Nov 27-38, 1975
IRIS SETTING	5.6 and 4.0
CONDITION OF BORING	Good
DEPTH PHOTOGRAPHED	35.0 to 220.0'
WATER DEPTH	Flowing at Surface
WATER CONDITION	Clear
FEET CASING (In Photo)	35.0-39.0'
FEET CONCRETE (In Photo)	None
FEET ROCK (In Photo)	39.0-220.0'

DEPTH RANGE	DESCRIPTION
45.5-46.2	Jt., Str. N 45 °E, dip 80 °NW, 1/8" at top to 1/32" at bottom, healed
	with white material (smooth), planar, terminates at bedding Jt. at bottom.
45.2-46.3	Gray-green rock
46.2	Bedding Jt., Str. N-S, dip 15 °E, 1/16" partly open, rough, planar
46.3-160.0	Dark gray rock containing numerous small irregular white inclusions
	At 51 feet rock gradually changes to dark blue-gray
	color.
53.6	Jt. Str. N 70 °E, dip 20 °SE, 1/32-1/16" partly open, stained, rough, planar
53.9-54.1	Jt., Str. N 20 °W, dip 30 °NE, 1/32-1/16" partly open, stained, rough,
	planar
54.3-54.7	Jt., Str. N 30 °W, dip 50 °NE, hairline-1/32", healed with white material,
	rough and irregular
56.2-56.3	Jt., Str. about N-S, dip 45 °W, 1/32", healed with white material, rough,
	irregular, discontinuous
56.7-57.9	Jt., Str. N 30 °E, dip 80 °NW, hairline-1/32", healed with white material,
	rough, planar, discontinuous
58.4-59.3	Jt., Str. N 10 °E, dip 75 °W, 1/32-1/16" healed with white material, rough,
	planar
59.1	Jt., Str. N-S, dip 10 °E, 1/16" healed with white material, rough, irregular
59.0-59.5	Jt., Str. N 10 °E, dip 75 °W, 1/16" healed with white material, rough,
	planar, discontinuous
59.7-61.5	3 Jts., Str. N 10 °E, dip 75 °W, 1/32-1/16" healed with white material,
	rough, planar
60.1-60.2	Jt., Str. N 70 °E, dip 30 °SE, 1/16-1/8" partly open, stained, rough,
	irregular
60.5-61.6	Jt., Str. N 10 °E, dip 75 °W, 1/16-1/8" healed with white material, rough,
	planar

PHOTOGRAPHED BY	LOGGED BY
Richard Hunt	Richard Hunt

ENG FORM 1 JUN 57 2391 (TRANSLUCENT) PREVIOUS EDITIONS ARE OBSOLETE

FIGURE B-1

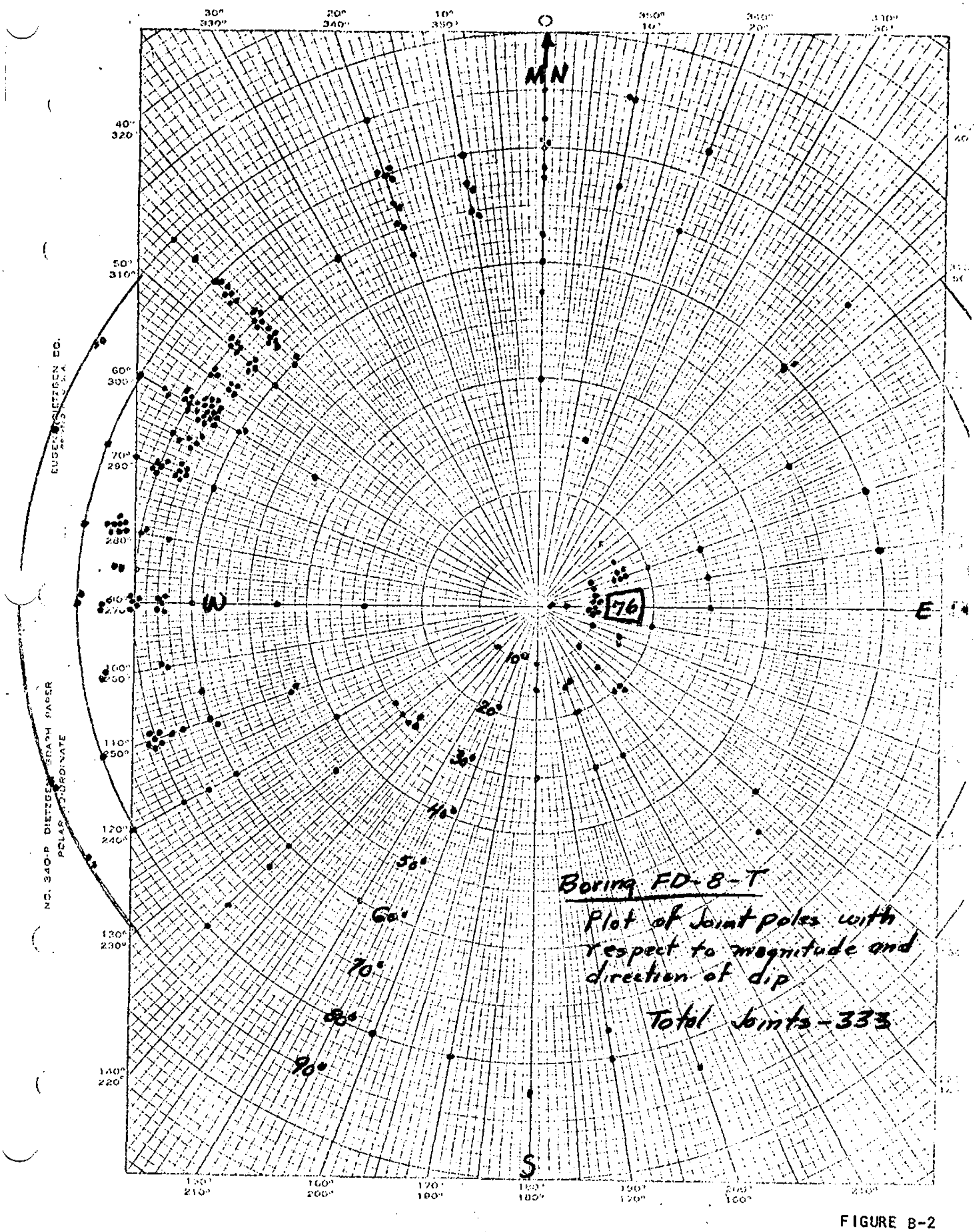

MN
W
E
S
10°
20°
30°
40°
50°
60°
70°
80°
90°
76
Boring FD-8-T
Plot of Joint poles with respect to magnitude and direction of dip
Total Joints-333
NO. 340-P DIETZGEN GRAPH PAPER
POLAR COORDINATE
EUGENE DIETZGEN CO.

FIGURE B-2

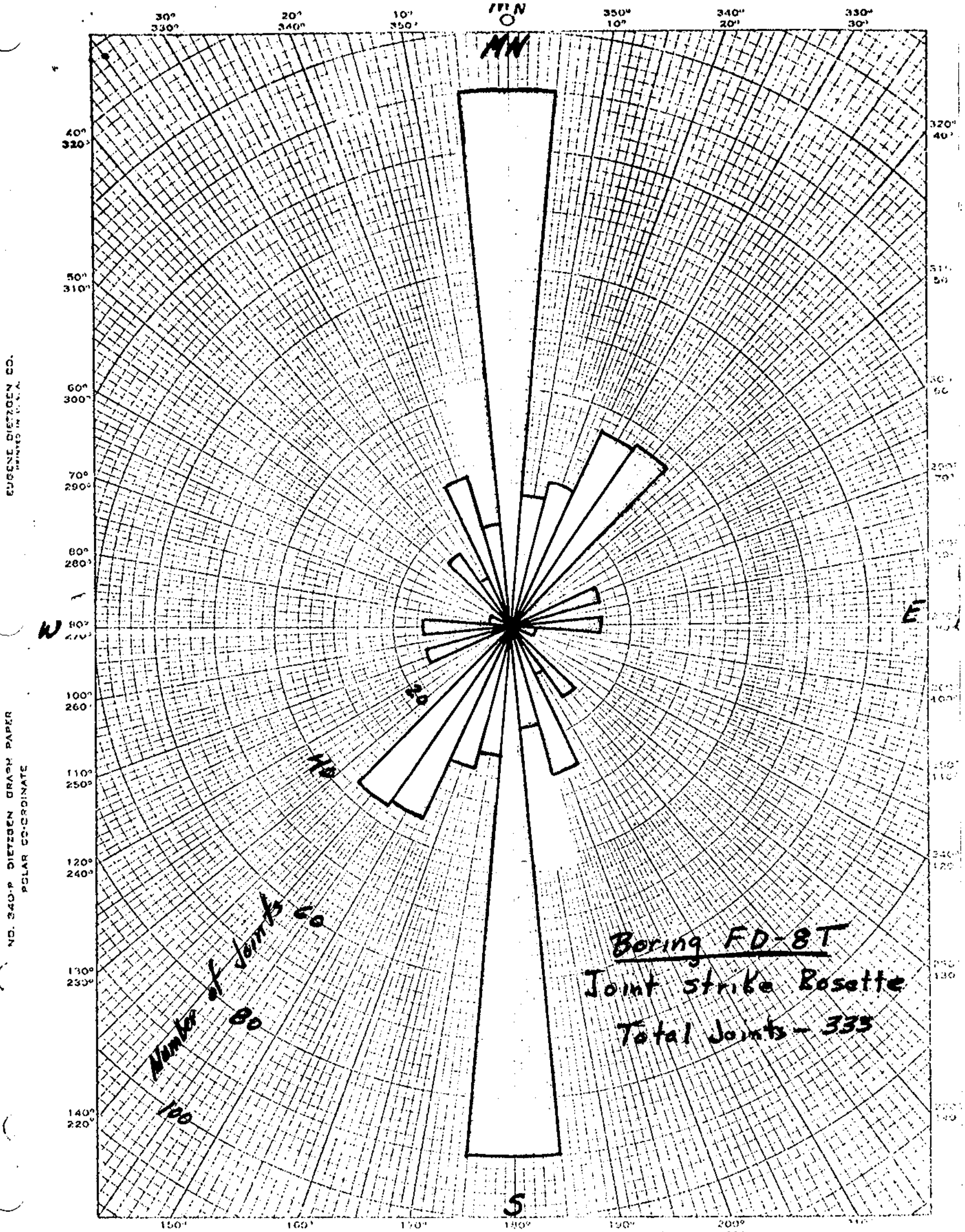


FIGURE B-3

Incl 3

APPENDIX C

WELL DATA AND PUMP TESTS

CONTENTS

TABLES

SUMMARY OF WATER LEVELS

BOREHOLE	2T	3T	4T	6T	9T	11T	12T	13T	13	15T	17T	18T (1)	19T	22T	25T	26T	27T	28T	29T	30T	31T
PIEZOMETER (PZ) OR OPEN WELL (OW)	PZ	PZ	PZ	PZ	PZ	PZ	OW	OW	PZ	PZ	OW	PZ/OW	OW	OW	OW	OW	OW	OW	OW	OW	OW
GROUND EL. (FT)	69.5	19.5	63.6	18.0	76.8	69.0	59.9	60.5	61.4	62.3	56.7	82.1	34.6	73.3	13	55	31	21.3	22	18.5	20.7
DEPTH TO PZ TIP OR BOTTOM	58.9	29	56.7	32.6	85.5	43	220.4	199.7	78	33.5	97	27.4/ 221	189.0	214.6	191.0	198.5	213.2	205.1	200.5	193.2	200.0
DEPTH TO ROCK (FEET)	35	52	17.6	31.0	80.5	37.7	16.4	33.5	60	28.8	25.7	6.0	21.3	28.3	80.0	55.2	92.1	85.5	69.0	54.0	75.0
STATION (OFFSET)	72+80 (30R)	21+92 (217R)	44+15 (366L)	28+31 (60R)	94+86 (35R)	38+62 (19L)	43+36 (30R)	48+79 (23L)	97+50 (48L)	51+88 (17L)	46+20 (17L)	62+56 (33R)	35+86 (20L)	57+55 (45L)	7+25	98+20	11+36 22R	13+50 17R	15+65 (35R)	23+10 58L	17+50 (13R)
DATE	**DEPTH TO WATER (FT)**																				
3/22/73	5.9	7.1																			
3/29/73	6.1	7.5	11.0																		
7/31/73	7.3	9.9	20.5																		
8/8/73	7.4	10.1	20.8																		
8/16/73	7.4	10.2	20.9																		
8/20/73	7.8	10.3	12.5																		
8/29/73	7.9	10.9	14.1																		
9/6/73	8.0	11.4	16.0																		
9/12/73	8.0	12.0	16.7																		
9/17/73	8.0	12.6	18.0																		
9/24/73	8.2	12.6	19.4																		
10/2/73	8.4	12.7	21.8																		
10/10/73	8.5	12.7	22.4																		
10/11/73									36.6												
12/13/73	6.9	12.1	27.3						34.6												
10/28/75	5.1	8.1	24.6	5.8	16.9	30.5	22.4	5.9	34.4	8.1	21.1	9.9		3.9							
11/4/75	5.4	9.4	24.4	7.1	16.9	30.5	24.0	6.1	34.2	9.7	22.2	10.2	3.5	4.5							
11/10/75	5.7	9.6	24.0	6.9	17.2	31.0	25.1	6.1	34.4	10.0	23.2	10.5	3.7	4.9							
12/30/75	5.3	9.5	24.0	6.6	17.2	30.4	24.1	5.4	33.7			8.6	3.3	4.3	4.8						
5/17/76	6.1	8.7	21.7	6.8	back-	30.7	24.9		33.7	10.7		11.4(1)	3.3	5.0	3.5	22.7					
6/4/76	6.2	10.2	21.7	7.2	filled	31.3	24.9	5.5	26.3	10.9	21.8	11.5	3.0	5.0	3.3		24.3				
7/22/76	7.5	16.0	23.2	13.4		33.6		7.6	34.1	12.3			11.0	6.0	3.2	29.4		17.8	17.2		
11/3 - 4/76	5.7	10.7	25.2	7.3		33.7	26.6	6.7	33.7	10.9		11.8	9.8	5.4	5.6	27.3	23.2	13.8	13.3	9.9	12.0

NOTES

1. Piezometer in original shallow borehole destroyed for borehole deepening.

TABLE C-2

SUMMARY OF PUMP TEST RESULTS

FD-	STATION	GROUND ELEVATION (FT)	DEPTH TO BEDROCK (FT)	DEPTH TO SCREEN (FT)	STATIC WATER DEPTH (FT)	GPM
12T	43+36	60	16	177	32.8	11
19T	35+86	34.6	21.3	160	47.5	12
25T	7+25	13	87	160	14.5	11
26T	98+20	55	55	150	66	9
27T	11+36	31	92	149	31	13
29T	15+65	22	69	155	21	13
30T	23+10	18.5	54	150	18.1	11

APPENDIX D

LABORATORY TESTING AND RESULTS

CONTENTS

TABLES

FIGURES

LABORATORY TEST PROCEDURES

Due to lack of well defined laboratory procedures, the following tests are briefly described below.

1. Sliding Friction on natural bedding joints were tested in a direct shear device using a cylindrical shear box adapted to accommodate "NX" diameter rock core. All sliding friction tests were conducted at a constant strain range of 0.008 to 0.012 in/min. and at normal stresses of 5, 10 and 15 T.S.F., and all open joint surfaces were in an immersed condition. For laboratory notes and typical test data see Figure D-2.

2. Multi-Stage Triaxial Tests on saturated undrained specimens were conducted in accordance with CRD-C 147-68 and as outlined in Appendix "C" of MRD First Interim Report, dated July 1966, titled, Strength Parameters of Selected Intermediate Quality Rocks. All cores were surface prepared to within required tolerances using Concinnati centerless and Pope standard surface grinding machines. Samples were tested at constant rates of strain and at varying confining pressures; total duration of these tests ranging from 2 to 20 minutes. For laboratory notes and typical test data see Figure D-4.

3. Modulus of Elasticity and Poisson's Ratio were computed from results of controlled strain measurements taken during unconfined compression testing data obtained separately on rock core specimens and from dynamic test data. In this connection, strain gages for the static determinations were applied to rock specimens primarily as outlined in Appendix "B" of MRD First Interim Report dated July 1966, titled, "Strength Parameters of Selected Intermediate Quality Rocks. Dynamic tests were performed using "Terrametrics" Sonic Velocity equipment and in accordance with the procedures furnished therewith.

4. Unconfined Compression Tests were conducted in accordance with the standard procedures.

SUMMARY OF TEST RESULTS																	
BOREHOLE NUMBER		FD-8T													FD-9T		
CORE DIAMETER		NX													NX		
DEPTH OF SAMPLE (FEET)		86-88 (S1)	86-88 (S2)	86-88 (S3)	93-96 (S1)	93-96 (S2)	93-96 ($S1_1$)	93-96 ($S1_2$)	93-96 ($S1_3$)	102-103 (S1)	102-103 (S2)	158.8-159.9	172.1-173	184.8-186.6	140.1-140.8	143.0-144.0	145.7-146.9
ROCK TYPE		BASALT	BASALT	BASALT	BASALT	BASALT	BASALT	BASALT	BASALT	BASALT	BASALT	BASALT	RED SHALE	RED SHALE	RED SANDSTONE	RED SANDSTONE	APHANITE
SPECIFIC GRAVITY @ 20° C	APPARENT	2.76	2.78	2.77	2.78	2.77	2.78	2.77	2.77	2.83	2.83	2.88	2.72		2.79	2.69	2.70
	BULK DRY	2.72	2.73	2.73	2.76	2.76	2.76	2.76	2.75	2.81	2.80	2.87	2.67		2.73	2.58	2.54
	BULK SSD	2.73	2.74	2.74	2.77	2.77	2.77	2.77	2.76	2.82	2.81	2.86	2.69		2.75	2.62	2.60
UNIT WEIGHT (pcf)		169.7	170.4	170.4	172.2	172.2	172.2	172.2	171.6	175.3	174.7	179.1	166.6		170.4	161.0	158.5
ABSORPTION %		0.5	0.4	0.4	0.7	0.8	0.7	0.8	0.8	0.3	0.4	0.30	0.8		0.80	1.6	2.30
SATURATION %		0	100	0	0	100	100	100	0	100	0	0	0			0	
DYNAMIC TESTING — SONIC VELOCITIES — COMP. WAVE	@ 300 psi				15,300	13,970										14,770	
	@ 600 psi				15,300	21,290										14,776	
	@ 900 psi				14,790	22,360										15,340	
SONIC VELOCITIES — SHEAR WAVE	@ 300 psi				4,570	2,210										4,340	
	@ 600 psi				4,570	4,380										4,340	
	@ 900 psi				4,670	4,860										4,750	
POISSON'S RATIO U	@ 300 psi				0.446	0.487										0.453	
	@ 600 psi				0.446	0.478										0.453	
	@ 900 psi				0.445	0.475										0.447	
SHEAR MODULUS G (psi x 10^6)	@ 300 psi				0.777	0.183										0.652	
	@ 600 psi				0.777	0.712										0.652	
	@ 900 psi				0.810	0.874										0.784	
MODULUS OF ELASTICITY E(psi x 10^6)	@ 300 psi				1.210	0.54										1.900	
	@ 600 psi				1.210	2.104										1,900	
	@ 900 psi				2.340	2.578										2,270	
UNCONFINED COMPRESSION (psi)												8,870		8,370	9,350	9,536	2,700 (1)
UNCONFINED COMPRESS MEASURED STRAIN	max (psi)	11,330	11,920	5,540			5,650	12,730	9,440	13,500	10,930		7,070				
	U	0.50	0.22	0.15			1.1	0.47	0.30	0.10	0.25		0.32				
	E(psi x10^6)	7.27	6.0	2.7			6.11	0.89	2.85	3.07	2.67		5.00				
TRIAXIAL SHEAR	E(psi x10^6)																
	U																
	psi																
MULTI-STAGE TRIAXIAL	ø																
	C(TSF)																
SLIDING FRICTION	ø																
	C(TSF)																

(1) Sample had natural fracture prior to application of axial load.

SUMMARY OF TEST RESULTS																
BOREHOLE NUMBER	FD-9T				FD-12T								FD-13T			
CORE DIAMETER	NX				4 IN.								NX			
DEPTH OF SAMPLE (FEET)	153.6-154.4	167.0-167.6			56.9-57.4	166.9-168.2	172.2 173.2	185-186					139.2-140.8	140.9 - 141.7	152.2-153.1	171.2-172.3
ROCK TYPE	APHANITE	APHANITE			RED SHALE	BLACK SHALE	RED SHALE	RED SHALE					RED SHALE	RED SHALE	RED SHALE	RED SHALE
SPECIFIC GRAVITY @ 20° C — APPARENT	2.68	2.71				2.70	2.72	2.73					2.73		2.73	2.72
SPECIFIC GRAVITY @ 20° C — BULK DRY	2.46	2.62				2.64	2.68	2.68					2.70		2.68	2.68
SPECIFIC GRAVITY @ 20° C — BULK SSD	2.54	2.65				2.66	2.70	2.70					2.71		2.70	2.69
UNIT WEIGHT (pcf)	153.5	163.5				167.2	167.2	167.2					168.5		167.2	167.2
ABSORPTION %	3.2	1.3											0.5		0.6	1.1
SATURATION %	0														0	
DYNAMIC TESTING — SONIC VELOCITIES — COMP. WAVE @ 300 psi	14,160														10,760	
@ 600 psi	14,160														12,100	
@ 900 psi	15,250														12,490	
SONIC VELOCITIES — SHEAR WAVE @ 300 psi	3,890														1,330	
@ 600 psi	4,310														1,450	
@ 900 psi	4,360														2,250	
POISSON'S RATIO U @ 300 psi	0.459														0.492	
@ 600 psi	0.449														0.493	
@ 900 psi	0.455														0.483	
SHEAR MODULUS G (psi x 10^6) @ 300 psi	0.500														0.063	
@ 600 psi	0.615														0.078	
@ 900 psi	0.629														0.183	
MODULUS OF ELASTICITY E(psi x 10^6) @ 300 psi	1.460														0.190	
@ 600 psi	1.780														0.230	
@ 900 psi	1.830														0.540	
UNCONFINED COMPRESSION (psi)		6,660				14,740							11,080			6,494
UNCONFINED COMPRESS MEASURED STRAIN — max (psi)	2,909						11,800								4,800	
U	0.10														0.12	
E(psi x10^6)	3.0														2.8	
TRIAXIAL SHEAR — E(psi x10^6)																
U																
psi																
MULTI-STAGE TRIAXIAL — ø																
C(TSF)																
SLIDING FRICTION — ø																
C(TSF)																
					(1)											

(1) Swell test performed.

SUMMARY OF TEST RESULTS

BOREHOLE NUMBER	FD-14T			FD-16T				FD-18T				FD-19T				
CORE DIAMETER	NX			NX				NX				4 IN.				
DEPTH OF SAMPLE (FEET)	222.4-224.7 (1)	222.4-224.7 (2)	222.4-224.7 (3)		65. - 66	78.0 - 78.6			35.0 - 35.4			32.7-33.0	35.7-36.0	36.7-37.1	129.4-131.5	144.7-145.6
ROCK TYPE	SHALE	SHALE	SHALE		RED SHALE	RED SHALE			RED SHALE			RED SHALE	RED SHALE	RED SHALE	GRAY SHALE	RED SHALE
SPECIFIC GRAVITY @ 20° C — APPARENT	2.67	2.69	2.67		2.77	2.72			2.78			2.79	2.78	2.78	2.66	2.73
SPECIFIC GRAVITY @ 20° C — BULK DRY	2.57	2.53	2.55		2.61	2.60			2.65			2.72	2.65	2.65	2.61	2.67
SPECIFIC GRAVITY @ 20° C — BULK SSD	2.60	2.53	2.57		2.67	2.65			2.69			2.72	2.69	2.69	2.63	2.69
UNIT WEIGHT (pcf)	160.4	157.9			162.9	162.2			165.4			174.1	173.5	173.5	162.9	166.6
ABSORPTION %	1.2	2.3	1.7		2.1	3.6			1.7			1.5	1.8	1.8	0.7	0.7
SATURATION %	0	0			0	0			0			0	0	0	0	0
DYNAMIC TESTING — SONIC VELOCITIES — COMP. WAVE @ 300 psi	20,930	19,650			10,160	9,800			9,480							
DYNAMIC TESTING — SONIC VELOCITIES — COMP. WAVE @ 600 psi	23,540	19,650			13,970	11,560			13,120							
DYNAMIC TESTING — SONIC VELOCITIES — COMP. WAVE @ 900 psi	23,540	17,860			13,970	11,870			13,120							
DYNAMIC TESTING — SONIC VELOCITIES — SHEAR WAVE @ 300 psi	2,670	4,320			3,520	3,550			2,490							
DYNAMIC TESTING — SONIC VELOCITIES — SHEAR WAVE @ 600 psi	2,750	4,270			3,720	3,760			3,410							
DYNAMIC TESTING — SONIC VELOCITIES — SHEAR WAVE @ 900 psi	2,900	4,680			3,920	3,960			3,710							
POISSON'S RATIO U @ 300 psi	0.492	0.475			0.431	0.442			0.464							
POISSON'S RATIO U @ 600 psi	0.493	0.475			0.460	0.441			0.475							
POISSON'S RATIO U @ 900 psi	0.492	0.463			0.460	0.488			0.457							
SHEAR MODULUS G ($psi \times 10^6$) @ 300 psi	0.248	0.634			0.435	0.433			0.215							
SHEAR MODULUS G ($psi \times 10^6$) @ 600 psi	0.262	0.622			0.509	0.485			0.405							
SHEAR MODULUS G ($psi \times 10^6$) @ 900 psi	0.291	0.745			0.539	0.538			0.478							
MODULUS OF ELASTICITY E ($psi \times 10^6$) @ 300 psi	0.740	1.870			1.245	1.249			0.630							
MODULUS OF ELASTICITY E ($psi \times 10^6$) @ 600 psi	0.780	1.840			1.486	1.348			1.195							
MODULUS OF ELASTICITY E ($psi \times 10^6$) @ 900 psi	0.870	2.180			1.574	1.655			1.393							
UNCONFINED COMPRESSION (psi)			12,256												4,329	
UNCONFINED COMPRESS MEASURED STRAIN — max (psi)									9,500			2,703	5,865			6,897
UNCONFINED COMPRESS MEASURED STRAIN — U									0.25			0.65	0.26			0.06
UNCONFINED COMPRESS MEASURED STRAIN — E ($psi \times 10^6$)									0.7			0.2	0.9			3.0
TRIAXIAL SHEAR — E ($psi \times 10^6$)																
TRIAXIAL SHEAR — U																
TRIAXIAL SHEAR — psi																
MULTI-STAGE TRIAXIAL — ϕ	36°															
MULTI-STAGE TRIAXIAL — C(TSF)	4															
SLIDING FRICTION — ϕ														27.3°		
SLIDING FRICTION — C(TSF)														0.55		

SUMMARY OF TEST RESULTS

BOREHOLE NUMBER		FD-19T	FD-20T			FD-22T							FD-23T				
CORE DIAMETER		4 IN.	NX	NX		4 IN.							NX				
DEPTH OF SAMPLE (FEET)		160.7-161.6	58.2-59.0	140.1-141.2	156.1-157.2	168.6-170.0	153 - 153.4	153.4-154.2	165.1-166	169.4-171.4	171.4-172.5	180 - 181.3					
ROCK TYPE		RED SHALE	RED SHALE	RED SHALE	GRAY SHALE	RED SHALE	BLACK SHALE	BLACK SHALE	RED SHALE	RED SHALE	RED SHALE	RED SHALE					
SPECIFIC GRAVITY @ 20° C	APPARENT	2.70	2.63		2.77	2.77	2.69			2.70		2.72					
	BULK DRY	2.64	2.58		2.73	2.73	2.65			2.67		2.68					
	BULK SSD	2.66	2.69		2.75	2.75	2.66			2.68		2.69					
UNIT WEIGHT (pcf)		164.7	161.0		170.4	170.4	165.4			166.6		167.2					
ABSORPTION %		0.9	0.7		0.6	0.5											
SATURATION %		0	0														
DYNAMIC TESTING — SONIC VELOCITIES — COMP. WAVE	@ 300 psi		6,930		18,470												
	@ 600 psi		13,860		18,470												
	@ 900 psi		16,380		17,630												
DYNAMIC TESTING — SONIC VELOCITIES — SHEAR WAVE	@ 300 psi		1,700		2,690												
	@ 600 psi		2,370		3,370												
	@ 900 psi		3,600		4,510												
POISSON'S RATIO U	@ 300 psi		0.468		0.489												
	@ 600 psi		0.485		0.483												
	@ 900 psi		0.435		0.465												
SHEAR MODULUS U (psi x 10^6)	@ 300 psi		0.105		0.266												
	@ 600 psi		0.195		0.419												
	@ 900 psi		0.451		0.748												
MODULUS OF ELASTICITY E(psi x 10^6)	@ 300 psi		0.298		0.790												
	@ 600 psi		0.598		1.240												
	@ 900 psi		1.294		2.190												
UNCONFINED COMPRESSION	(psi)	8,322		8,920		6,270		4,575				11,340					
UNCONFINED COMPRESS MEASURED STRAIN	max (psi)		13,100		10,580												
	U				0.15												
	E(psi x10^6)		0.4		2.50												
TRIAXIAL SHEAR	E(psi x10^6)																
	U																
	psi																
MULTI-STAGE TRIAXIAL	ø																
	C(TSF)																
SLIDING FRICTION	ø																
	C(TSF)																

		SUMMARY OF TEST RESULTS															
BOREHOLE NUMBER		FD-24T					FD-25T							FD-26T		FD-27T	
CORE DIAMETER		4 IN.					4 IN.							4 IN.			
DEPTH OF SAMPLE (FEET)		153.9-155.5 (S1)	153.9-155.5(S2)	155.1-157.1	161.0-163.5	165.0-166.6	91.1-92.0	121.9-123.1	129.6 130.9	131.0-132.0	138.7-139.7	155.7-157.0	168.0-169.1	99.3-100.3			
ROCK TYPE		BASALT	BASALT	BASALT	BASALT	BASALT	RED SHALE	RED SHALE	RED SHALE	RED SHALE	RED SHALE	RED SHALE	RED SHALE	RED SHALE			
SPECIFIC GRAVITY @ 20° C	APPARENT	2.77		2.77	2.77	2.77	2.73	2.73	2.69	2.71	2.69	2.73	2.75				
	BULK DRY	2.72		2.72	2.74	2.68	2.68	2.66	2.64	2.65	2.62	2.66	2.65				
	BULK SSD	2.71		2.71	2.75	2.70	2.70	2.68	2.66	2.67	2.65	2.68	2.68				
UNIT WEIGHT (pcf)		169.7		169.7	171.0	167.2	167.2	166.0	164.7	165.4	163.5	166.0	165.4				
ABSORPTION %		0.72		0.72	0.46	0.49	0.8	0.9	0.7	0.9	0.9	0.9	1.4				
SATURATION %		0															
DYNAMIC TESTING — SONIC VELOCITIES — COMP. WAVE	@ 300 psi																
	@ 600 psi																
	@ 900 psi																
DYNAMIC TESTING — SONIC VELOCITIES — SHEAR WAVE	@300psi																
	@ 600 psi																
	@ 900 psi																
POISSON'S RATIO U	@ 300 psi																
	@ 600 psi																
	@ 900 psi																
SHEAR MODULUS G (psi x 10^6)	@ 300 psi																
	@ 600 psi																
	@ 900 psi																
MODULUS OF ELASTICITY (psi x 10^6)	@ 300 psi																
	@ 600 psi																
	@ 900 psi																
UNCONFINED COMPRESSION (psi)		13,740	9,240				6,340	9,740					3,242				
UNCONFINED COMPRESS MEASURED STRAIN	max (psi)		9,134									5,441					
	U		0.29									0.35					
	E(psi x 10^6)		10.0									2.0					
TRIAXIAL SHEAR	E(psi x 10^6)																
	U																
	psi																
MULTI-STAGE TRIAXIAL	ϕ									23.4°				32.1°			
	C(TSF)									190				96			
SLIDING FRICTION	ϕ																
	C(TSF)																

		SUMMARY OF TEST RESULTS															
BOREHOLE NUMBER			FD-28T				FD-29T				FD-30T					FD-31T	
CORE DIAMETER																4 IN.	
DEPTH OF SAMPLE (FEET)																	188.4 - 189.5
ROCK TYPE																	RED SHALY SANDSTONE
SPECIFIC GRAVITY @ 20° C	APPARENT																
	BULK DRY																
	BULK SSD																
UNIT WEIGHT (pcf)																	
ABSORPTION %																	
SATURATION %																	
DYNAMIC TESTING — SONIC VELOCITIES — COMP. WAVE	@ 300 psi																
	@ 600 psi																
	@ 900 psi																
DYNAMIC TESTING — SONIC VELOCITIES — SHEAR WAVE	@ 300 psi																
	@ 600 psi																
	@ 900 psi																
DYNAMIC TESTING — POISSON'S RATIO U	@ 300 psi																
	@ 600 psi																
	@ 900 psi																
DYNAMIC TESTING — SHEAR MODULUS U (psi x 10^6)	@ 300 psi																
	@ 600 psi																
	@ 900 psi																
DYNAMIC TESTING — MODULUS OF ELASTICITY E(psi x 10^6)	@ 300 psi																
	@ 600 psi																
	@ 900 psi																
UNCONFINED COMPRESSION (psi)																	12,450
UNCONFINED COMPRESS MEASURED STRAIN	max (psi)																
	U																
	E(psi x 10^6)																
TRIAXIAL SHEAR	E(psi x 10^6)																
	U																
	psi																
MULTI-STAGE TRIAXIAL	ϕ																
	C(TSF)																
SLIDING FRICTION	ϕ																
	C(TSF)																

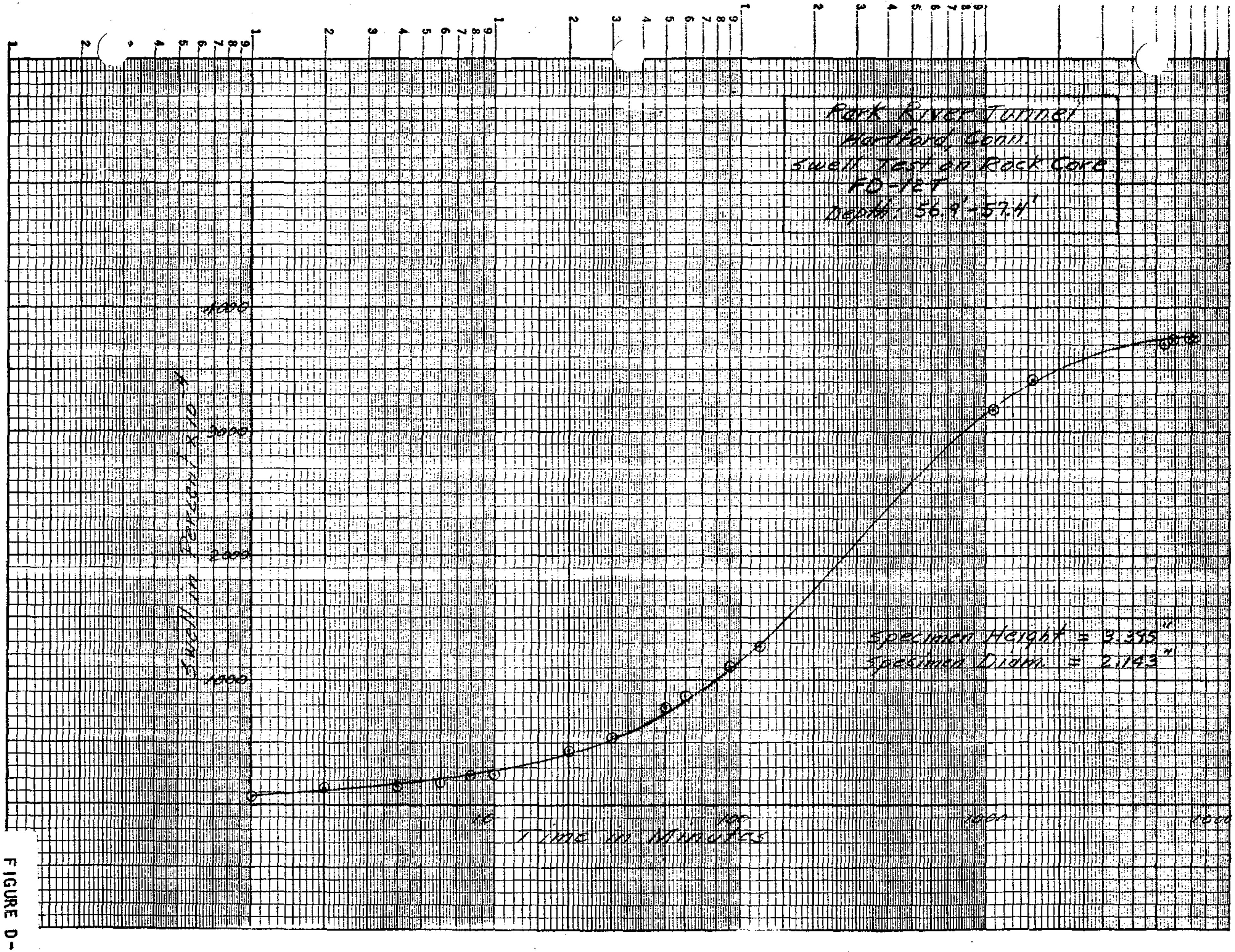
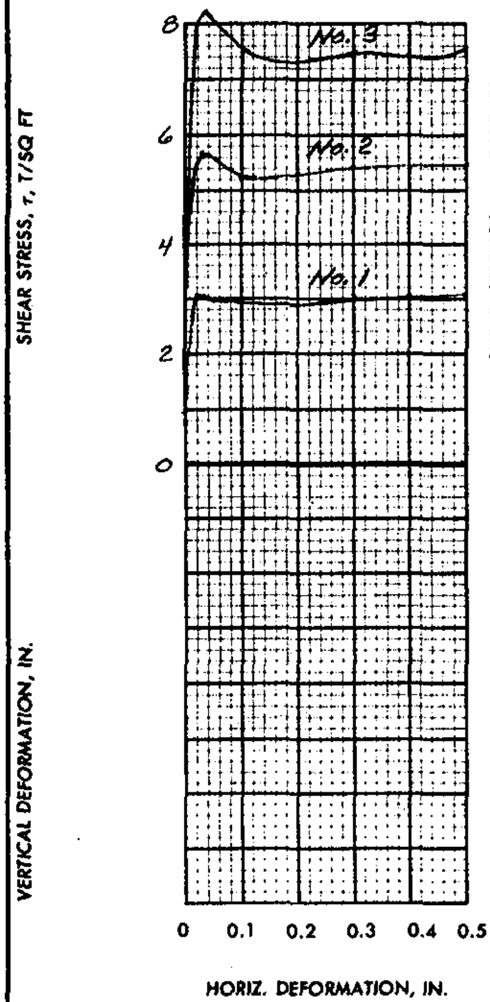
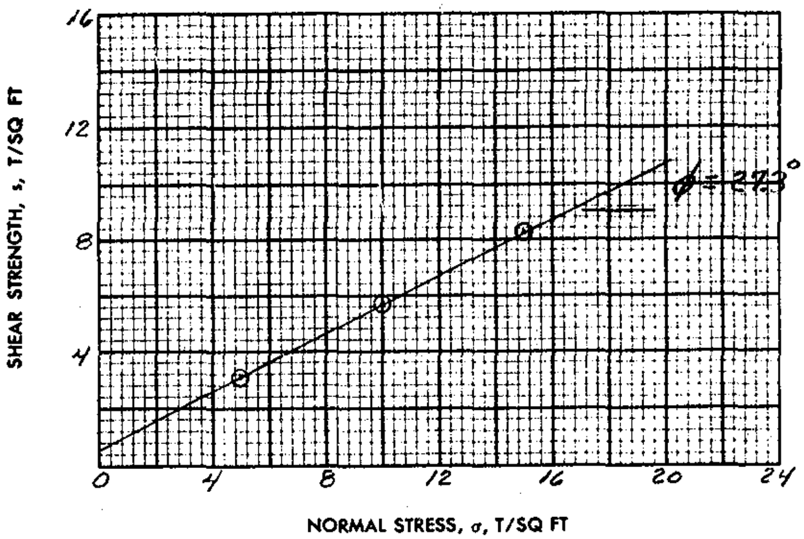

Park River Tunnel
Hartford Conn.
Swell Test on Rock Core
FD-127
Depth: 56.9'-57.4'
Specimen Height = 3.355"
Specimen Diam. = 2.143"
Swell in Percent X 10-4
1000
2000
3000
4000
10
100
1000
Time in Minutes

FIGURE D-1





TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	%	%	%	%
	VOID RATIO	e_o				
	SATURATION	S_o	%	%	%	%
	DRY DENSITY, LB/CU FT	γ_d				
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}				
FINAL	WATER CONTENT	w_f	%	%	%	%
	VOID RATIO	e_f				
	SATURATION	S_f	75+ %	75+ %	75+ %	%
NORMAL STRESS, T/SQ FT		σ	5.0	10.0	15.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	3.15	5.71	8.26	
ACTUAL TIME TO FAILURE, MIN		t_f	2.4	3.0	4.4	
RATE OF STRAIN, IN./MIN			0.008	0.008	0.008	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}	2.88	5.23	7.39	

SHEAR STRENGTH PARAMETERS

ϕ' = 27.3°

TAN ϕ' = 0.516

c' = 0.55 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TYPE OF SPECIMEN: Rock Core — 2.125 IN. ~~SQUARE~~ Diam. — IN. THICK

CLASSIFICATION: Sliding Friction on Natural Open Joint

LL	PL	PI		G.

REMARKS (1) Same specimen under different normal loads used throughout series.
(2) Cohesion value is "No-Load" strength of rock combination and results from changing roughness of interface between the 2 halves.

PROJECT: Park River Auxiliary Conduit

AREA

BORING NO. FD-19 — SAMPLE NO.

DEPTH ~~EL~~ 36.4' - 37.1' — DATE July 1976

DIRECT SHEAR TEST REPORT

FIGURE D-2

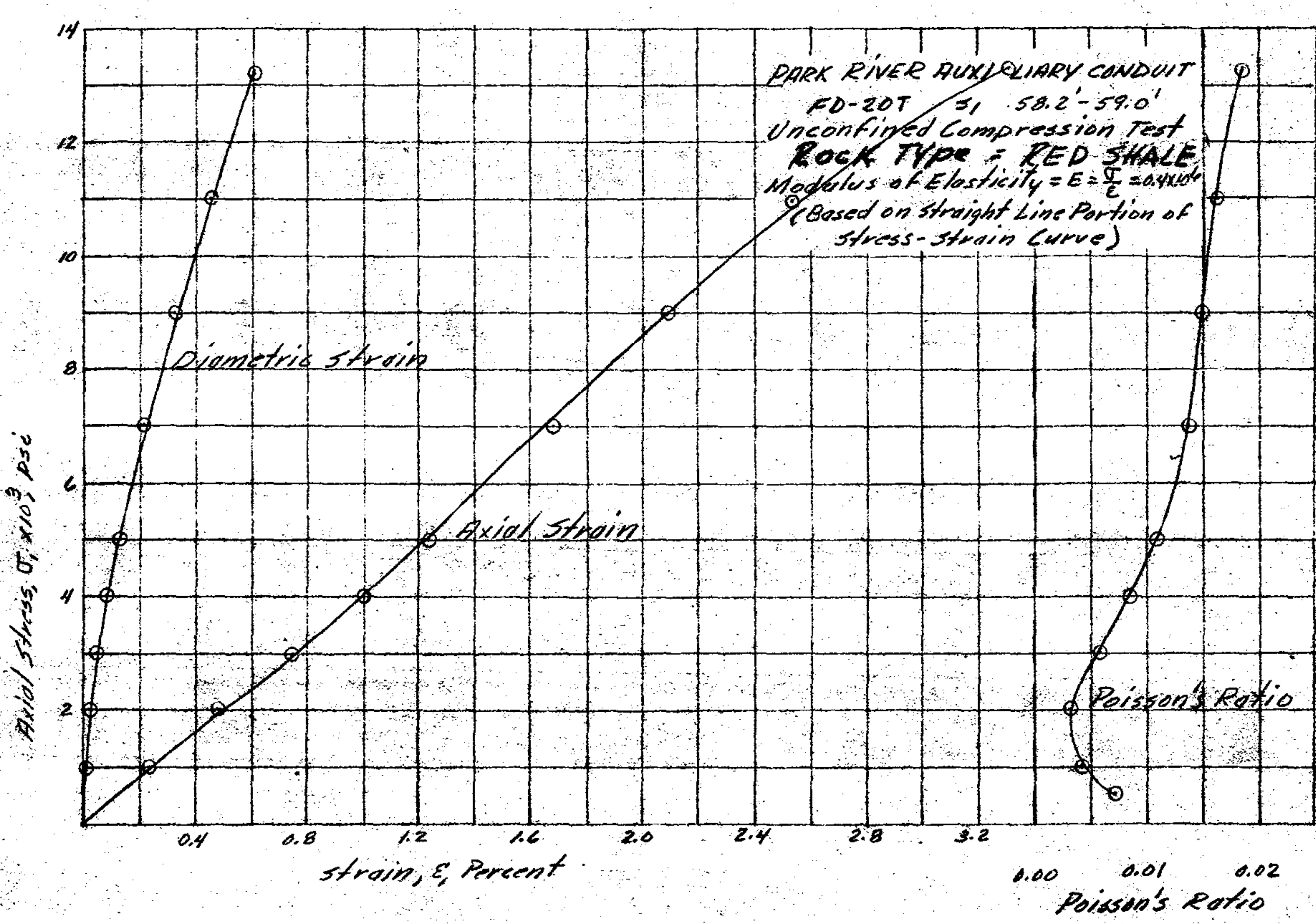

PARK RIVER AUXILIARY CONDUIT
FD-20T 31 58.2'-59.0'
Unconfined Compression Test
ROCK TYPE = RED SHALE
Modulus of Elasticity = E = σ/ε = 0.4x10⁶
(Based on Straight Line Portion of Stress-Strain Curve)
Diametric Strain
Axial Strain
Poisson's Ratio
Axial Stress, σ1 x10³ psi
Strain, ε, Percent
0.4
0.8
1.2
1.6
2.0
2.4
2.8
3.2
0.00
0.01
0.02
2
4
6
8
10
12
14

FIGURE D-3

SUBJECT Park River
COMPUTATION Precast Tunnel
COMPUTED BY JF CHECKED BY D.D. DATE Nov. '76

TRANSFORMED AREA DESIGN CASES

CASE 1
N outside section, $e_o > 0$

CASE 2
N inside section but outside Kern $e_o < 0$

CASE 3
N inside Kern, Min. Reinf.

CORRELTION FACTOR (INSIDE FACE ONLY)

FORMULAS FOR STRESS AND STRAIN by Ray Roark pg. 164 TABLE VII CASE 1

$$F = k_i = 1 + 0.5 \frac{I}{b c^2}\left[\frac{1}{R-c} + \frac{1}{R}\right]$$

$$F = 1 + \frac{0.5 \times 0.083}{1.0 \times 0.375^2}\left[\frac{1}{11.375-0.375} + \frac{1}{11.375}\right]$$

$F = 1.02$ (apply only to inside face)

SOLVE FOR S by trial and error

$$\tfrac{1}{2} b f_c S(S/3 + e_o) + \left(\frac{S-d'}{S}\right)(2n-1) f_c A_s'(d' + e_o) - \left(\frac{d-S}{S}\right) n f_c A_s (d + e_o) = 0$$

$$f_c = \frac{SNF^*}{\frac{bS^2}{2} + (S-d')(2n-1)A_s' + (d-S)\,n\,A_s}$$ CASE 1 & 2

$$f_s = \left(\frac{d-S}{S}\right) n f_c F^{**}$$ (TENSION STEEL) CASE 1 & 2

$$f_s' = \frac{(S-d')}{S} 2n f_c F^*$$ (COMP. STEEL) CASE 1 & 2

$$f_c = \frac{N}{bD} + \frac{6M}{bD^2}$$ CASE 3

Use MIN. REINF. FOR CASE 3
* APPLY WHEN M is POS.
* APPLY WHEN M is NEG.

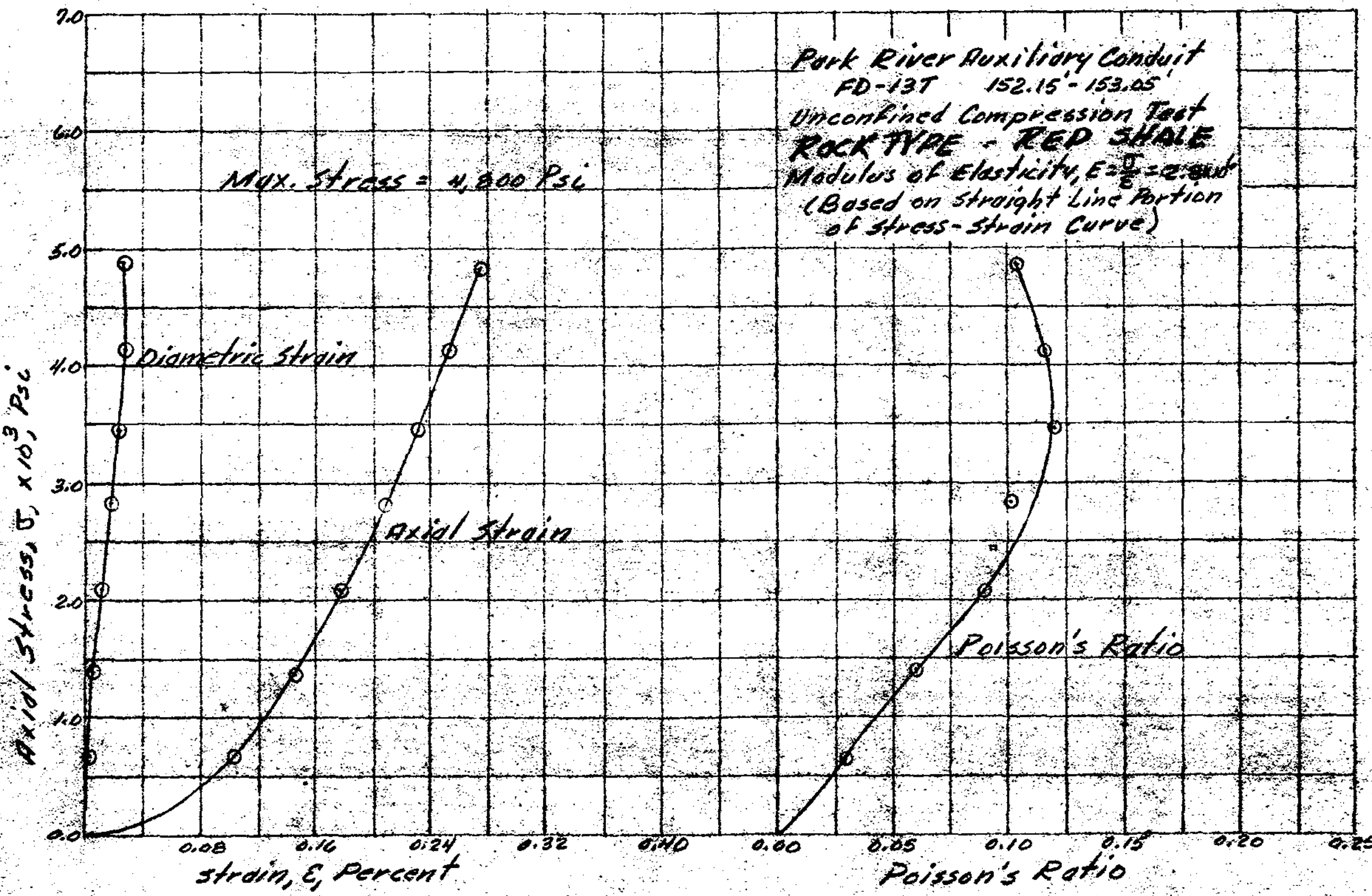
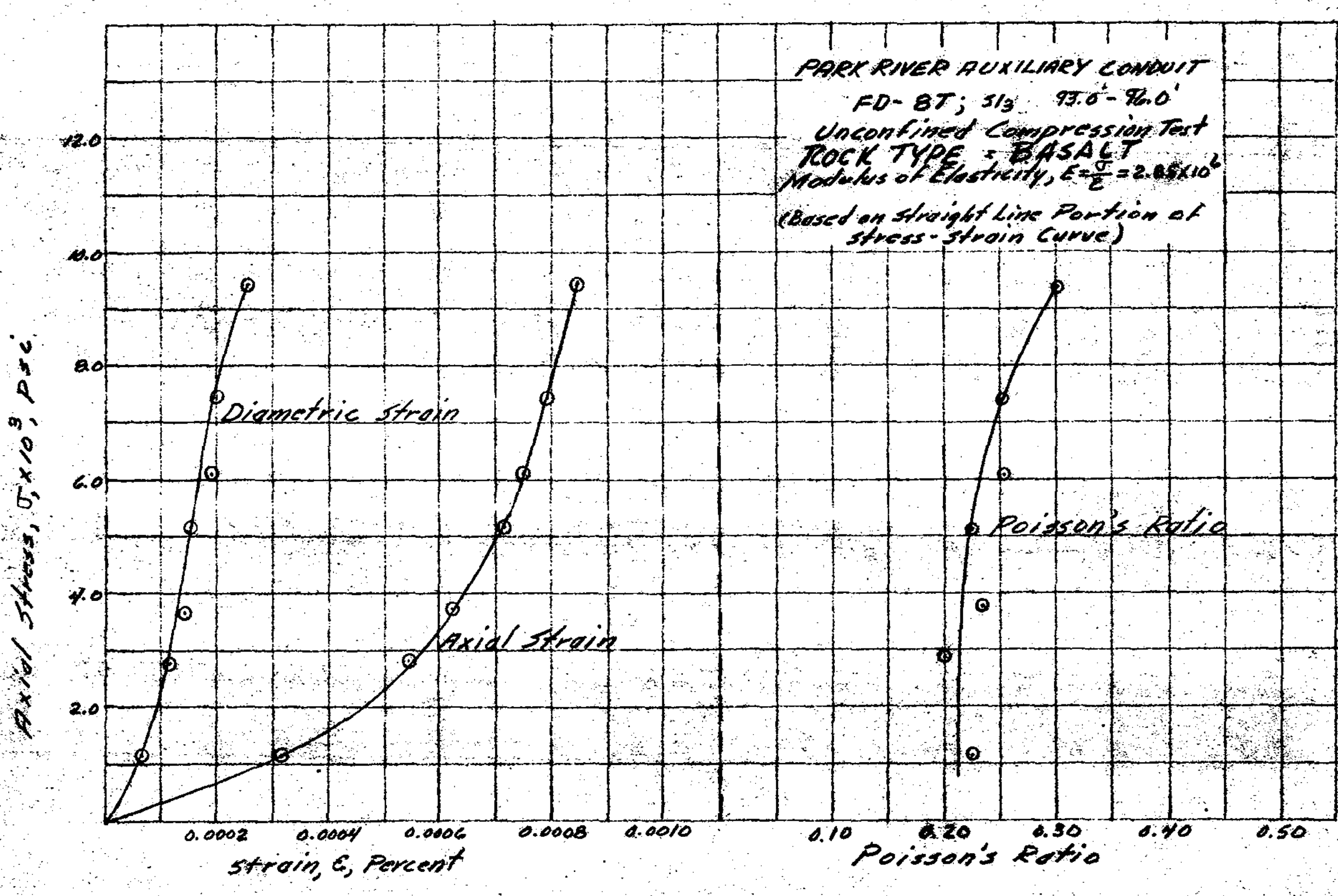

Park River Auxiliary Conduit
FD-13T 152.15' - 153.05'
Unconfined Compression Test
ROCK TYPE - RED SHALE
Modulus of Elasticity, E = σ/ε
(Based on Straight Line Portion of Stress-Strain Curve)
Max. Stress = 4,800 Psi
Diametric Strain
Axial Strain
Poisson's Ratio
Axial Stress, σ, x10³, Psi
Strain, ε, Percent
Poisson's Ratio
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
0.08
0.16
0.24
0.32
0.40
0.00
0.05
0.10
0.15
0.20
0.25

FIGURE D-5



PARK RIVER AUXILIARY CONDUIT
FD- 8T; 51/3 93.0'-96.0'
Unconfined Compression Test
ROCK TYPE = BASALT
Modulus of Elasticity, E=σ/ε=2.85x10^6
(Based on Straight Line Portion of stress-strain Curve)
Diametric strain
Axial Strain
Poisson's Ratio
12.0
10.0
8.0
6.0
4.0
2.0
0.0002
0.0004
0.0006
0.0008
0.0010
0.10
0.20
0.30
0.40
0.50
Axial Stress, σ, x10^3, psi
strain, ε, Percent
Poisson's Ratio

FIGURE D-6

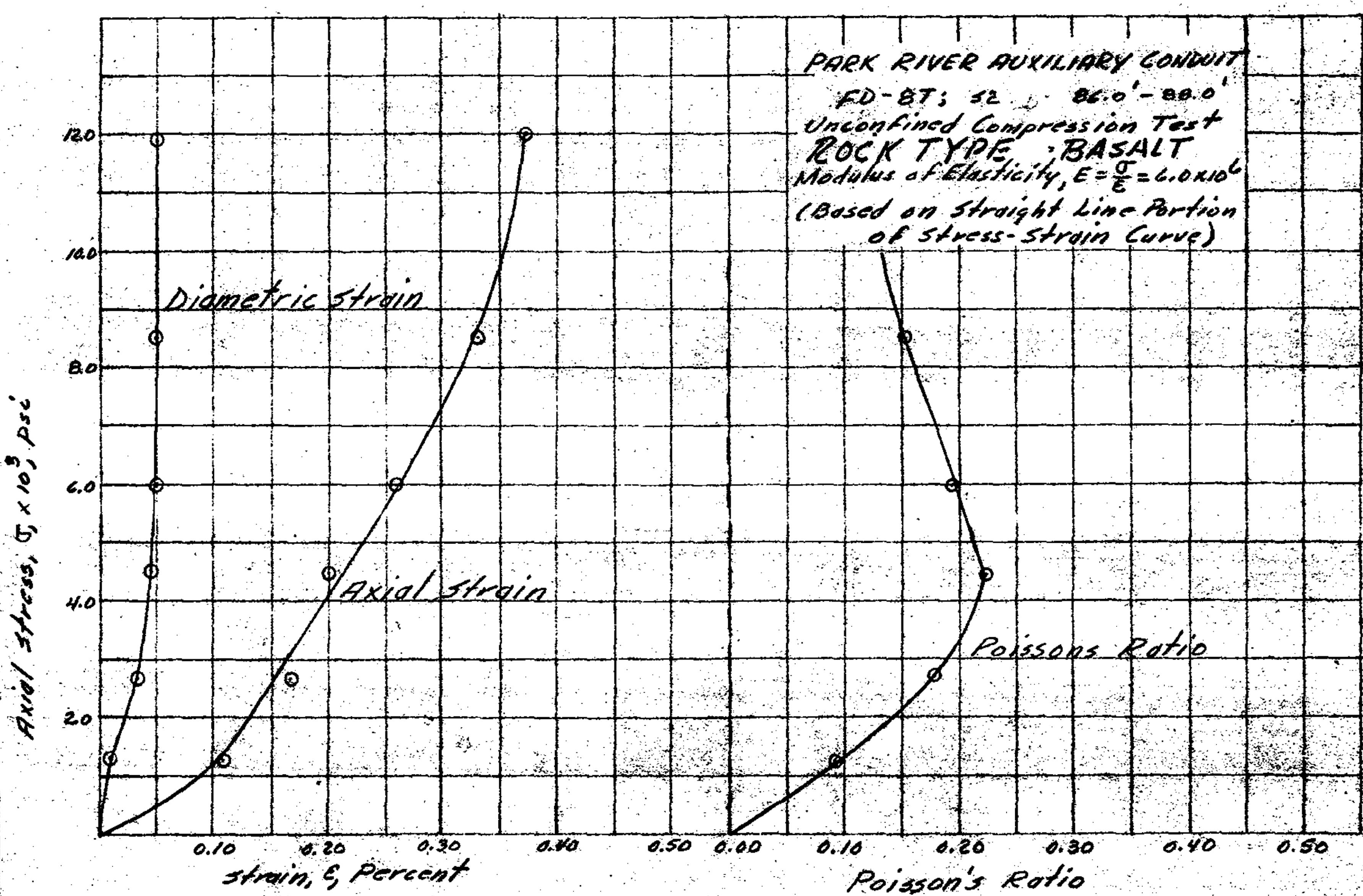

PARK RIVER AUXILIARY CONDUIT
FD-8T; S2 86.0'-88.0'
Unconfined Compression Test
ROCK TYPE : BASALT
Modulus of Elasticity, $E = \frac{\sigma}{\epsilon} = 6.0 \times 10^6$
(Based on Straight Line Portion of Stress-Strain Curve)
Diametric Strain
Axial Strain
Poissons Ratio
Axial Stress, σ, $\times 10^3$, psi
12.0
10.0
8.0
6.0
4.0
2.0
0.10
0.20
0.30
0.40
0.50
Strain, ϵ, Percent
0.00
0.10
0.20
0.30
0.40
0.50
Poisson's Ratio

FIGURE D-7

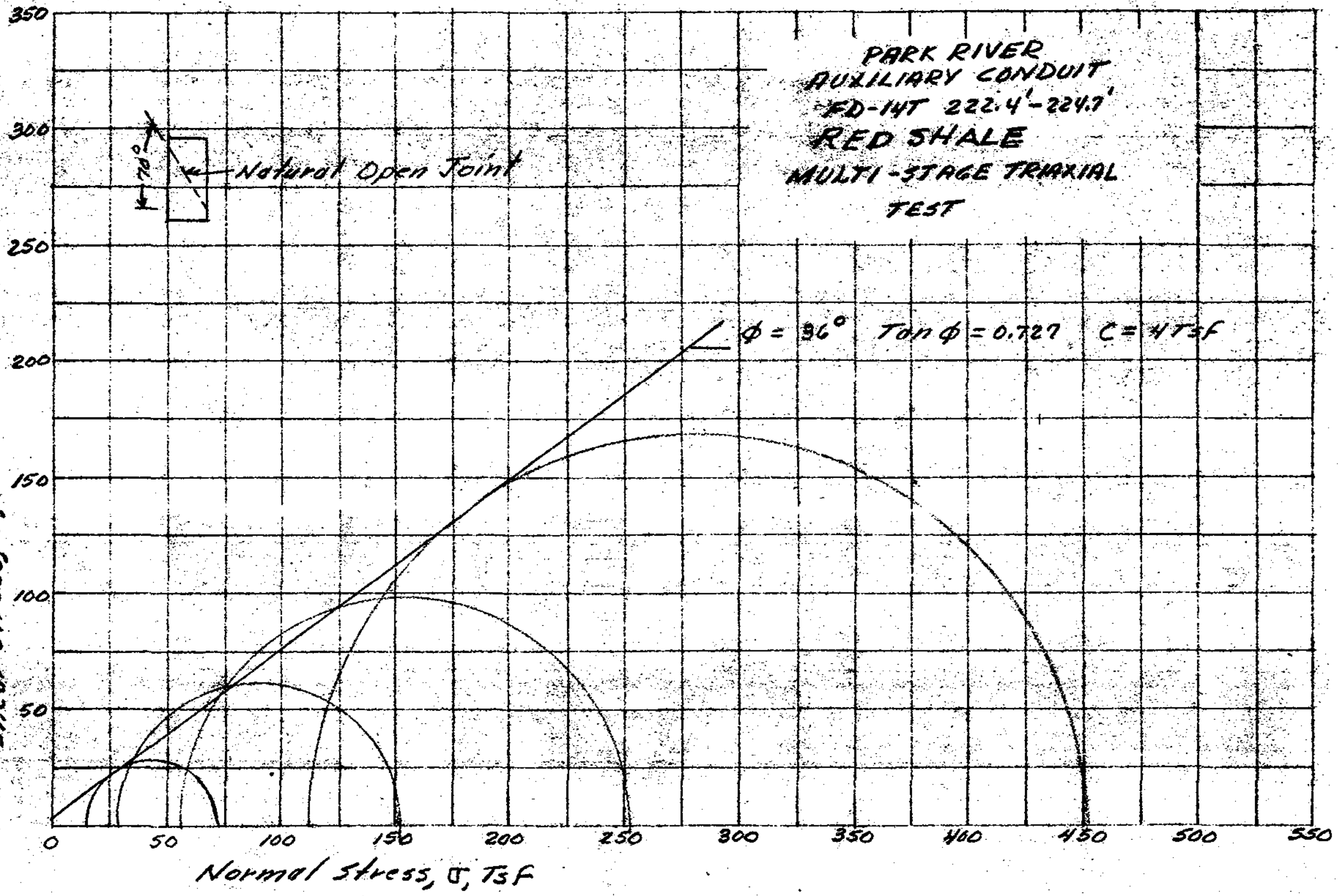
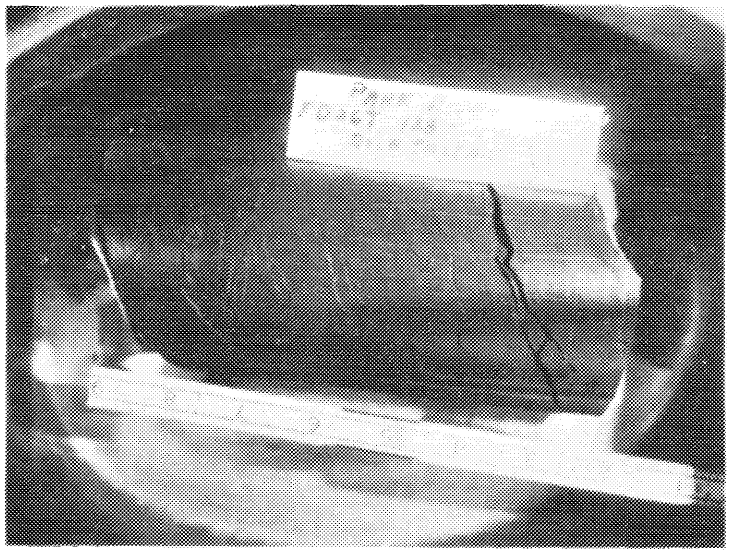

PARK RIVER
AUXILIARY CONDUIT
FD-14T 222.4'-224.7'
RED SHALE
MULTI-STAGE TRIAXIAL
TEST
70°
Natural Open Joint
φ = 36° Tan φ = 0.727 C = 4 TSF
Shear Stress, τ, TSF
Normal Stress, σ, TSF
0
50
100
150
200
250
300
350
400
450
500
550

FIGURE D-8



SUBMERGED RED SHALE CORE
SHOWING STRESS RELIEF

FIGURE D-9

APPENDIX E

COMPUTATIONS FOR ROCK LOADS AND GROUND WATER

CONTENTS

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Rock Load Determination
COMPUTATION by Barton Lein & Lunde Methods
COMPUTED BY ______ CHECKED BY ______ DATE 11/9/76

For Best Average Condition

Assumptions for ground Conditions

RQD = 80
Joint Sets = Bedding, Strike & Dip (Jn = 9)
Joint Roughness = Rough, & planar (Jr = 1.5)
Joint Alteration = Unaltered surface staining (Ja = 1.0)
Water Inflow = Minor inflow < 5 L/Min locally (Jw = 1.0)
Stress Reduction Factor = Medium Stress (SRF = 1)

Determination of Q = Rock Mass Quality

where $Q = RQD/J_n \cdot J_r/J_a \cdot J_w/SRF$

$= 80/9 \cdot 1.5/1.0 \cdot 1.0/1.0$

$= \boxed{13.35}$

Determination of P_{roof} = permanent roof support pressure Kg/cm²

where $P_{roof} = \dfrac{2 J_n^{1/2} (Q)^{-1/3}}{3 J_r}$

$= \dfrac{2 \cdot (9)^{1/2} \cdot (13.35)^{-1/3}}{3 \cdot 1.5}$

$= \boxed{.52}$ Kg/cm²

Design of Support (by Case Record)

where ESR = Excavation Support Ratio = 1.6
Span = 26.0' (7.92 m)

Span/ESR = 4.95 — Case 9-13

by Chart Support Required is

Untensioned bolts @ 6.0' c.c. to spot bolting

Determination of Bolt Length

where L = Length in meters, B = Span in meters

$L = 2 + 0.15\, B/ESR$

$L = 2.74\,m = \boxed{8.9\,ft}$

SUBJECT Rock Load Determination
COMPUTATION by Barton Lein and Lunde Method
COMPUTED BY ______ CHECKED BY ______ DATE 11/9/76

For. Worst Average Condition

Assumptions for Ground Conditions

RQD = 40
Joint Sets = Bedding Strike, dip ($J_n = 9$)
Joint Roughness = Rough - Planar ($J_r = 1.5$)
Joint Alteration = Slightly altered Joint Walls clay free ($J_a = 2.0$)
Water In flow = Medium w/occasional Joint outwash ($J_w = .66$)
Stress Reduction Factor = Medium Stress (SRF = 1)

Determination of Q = Rock Mass Quality

where $Q = RQD/J_n \cdot J_r/J_a \cdot J_w/SRF$

$= 40/9 \cdot 1.5/2.0 \cdot .66/1.0$

$= \boxed{2.19}$

Determination of P_{roof} = Permanent Roof Support Pressure TSF

where $P_{roof} = \dfrac{2 J_n^{1/2} (Q)^{-1/3}}{3 J_r}$

$= 2 \cdot (9)^{1/2} \cdot (2.19)^{-1/3}$

$= \boxed{1.02}$ TSF

Design of Support (by Case Record)

Where ESR = Excavation Support Ratio
Span = 26.0 ft (7.92 m)

Span/ESR = 4.95 Case 21-22

by Chart Support Required is

Untensioned bolts 3' C.C. w/ 1" Shotcrete

Determination of Bolt Lengths

Where L = Length in meters B = Span in meters

$L = 2 + 0.15 B/ESR = 2.74\ m = \boxed{8.9\ ft}$

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Rock Load Determination
COMPUTATION by Barton, Lein and Lunde Method
COMPUTED BY ________ CHECKED BY ________ DATE 11/9/96

For Fault Zones

Assumptions for Ground Conditions

RQD = 30

Joint Sets = Four or more, heavily Jointed ($J_n = 15$)

Joint Roughness = Rough or irregular, planar ($J_r = 3.0$)

Joint Alteration = Slightly altered joint walls clay free ($J_a = 2$)

Water Inflow = Large Inflow ($J_w = .5$)

Stress Reduction Factor = Multiple Shear Zones, Loose surrounding rock (SRF = 7.5)

Determination of Q = Rock Mass Quality

Where $Q = RQD/J_n \cdot J_r/J_a \cdot J_w/SRF$

$= 30/15 \cdot 1.5/2 \cdot .5/7.5$

$= \boxed{.1}$

Determination of P_{roof} = Permanent Roof Support Pressure TSF

where $P_{roof} = \dfrac{2 J_n^{1/2} (Q)^{-1/3}}{3 J_r}$

$= \dfrac{2 \cdot (15)^{1/2} (.1)^{-1/3}}{9}$

$= \boxed{1.85}$ TSF

Design of Support (by Case Record)

where ESR = Excavation Support Ratio

Span = 26.0 ft (7.92 m)

Span/ESR = 4.95 Cat. 31 Case (48)

$\boxed{\text{Tensioned bolts 3.0' c.c. w/wire mesh + 2 to 5" Shotcrete}}$

Determination of Bolt Lengths

where L = Length in meters B = Span in meters

$L = 2 + 0.15\, B/ESR = 2.74\text{ m} = \boxed{8.9\text{ ft}}$

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

PAGE E-4

SUBJECT: ROCK LOAD DETERMINATION

COMPUTATION: by Modified Terzaghi Method EM 1110-2-2901

COMPUTED BY ______ CHECKED BY ______ DATE 10/20/76

Best Average
For RQD = 80

Condition of Ground = Massive, moderately Jointed

Initial Load = 0

Final Load = $\frac{0.5B \times \gamma}{2000}$ when B = width (26')

$= \boxed{1.09}$ TSF

γ = 166 - 171 #c.f.

Avg = 169 #c.f.

Worst Average

For RQD = 40

Condition of Ground = very blocky, seamy

Initial Load = 0.5C when C = width + height

Final Load = 1.5C

$= \frac{0.5 \times 2B \times \gamma}{2000}$

$= \boxed{2.2}$ TSF

Fault Zone

For RQD = 30

Condition = Completely Crushed

Final Load = 1.1 C when C = width + height

$= \frac{1.1 \times 2B \times \gamma}{2000}$

$= \boxed{4.8}$ TSF

SUBJECT Ground Water Inflows During Tunnel Driving

COMPUTATION Ref: AEG Bulletin No. 1 Vol 2, 1965

COMPUTED BY YY CHECKED BY ______ DATE 12/9/76

Park River Auxiliary Conduit

Predicted inflows based on permeability (K) and inflow (q)

$K = 8q/(2\pi l \Delta p)$ and $q = \frac{2k(h+H)}{2.3 \log(r/2h)}$ (No drawdown of GWS)

K = permeability in ft/min; q = pumping rate; l = length of test section; Δp = gage pressure (corrected); h = distance from tunnel to ground; H = depth of standing water; r = tunnel radius (with overbreak)

Best Average Condition (RQD = 80, depth to GWS = 10 ft, q = 1 gpm)

- Δp = max. static head + gage pressure − friction loss
  = (10 ft × .43 psi/ft) + 50 psi − (.20)(50 psi) = 44 psi
  = 6336 psf.

- q = 1 gpm = .1337 ft³/min = 8.34 lb/min (of water)

$K = 8q/2\pi l\Delta p = (8)(8.34\ lb/min)/2\pi(5\ ft)(6336\ lb/ft^2)$

$K = 3.35 \times 10^{-4}$ ft/min.

$q = \frac{(2)(3.35\times10^{-4}\ ft/min)(170\ ft + 160\ ft)}{2.3 \log(13\ ft/(2)(170\ ft))} = -6.78\times10^{-2}\ ft^2/min$

q = 0.51 gpm (per foot of tunnel)

Worst Average Condition (in Leached Zones)

Assume depth = 130 ft, q = 15 gpm, h = 130 ft, H = 120 ft

- Δp = (10 ft × .43 psi/ft) + (50 psi) − (.20)(50 psi) = 44 psi

- q = 15 gpm = (15)(8.34 lb/min) = 125.1 lb/min.

$K = 8q/2\pi l\Delta p = (8)(125.1\ lb/min)/2\pi(5\ ft)(6336\ psf)$

$K = 5.0 \times 10^{-3}$ ft/min.

$q = \frac{2K(h+H)}{2.3\log(r/2h)} = \frac{(2)(5\times10^{-3}\ ft/min)(130\ ft + 120\ ft)}{2.3 \log(13\ ft/(2)(130\ ft))}$

q = −0.84 ft²/min = 6.2 gpm/ft (of tunnel)

APPENDIX F

TYPICAL STRUCTURAL COMPUTATIONS

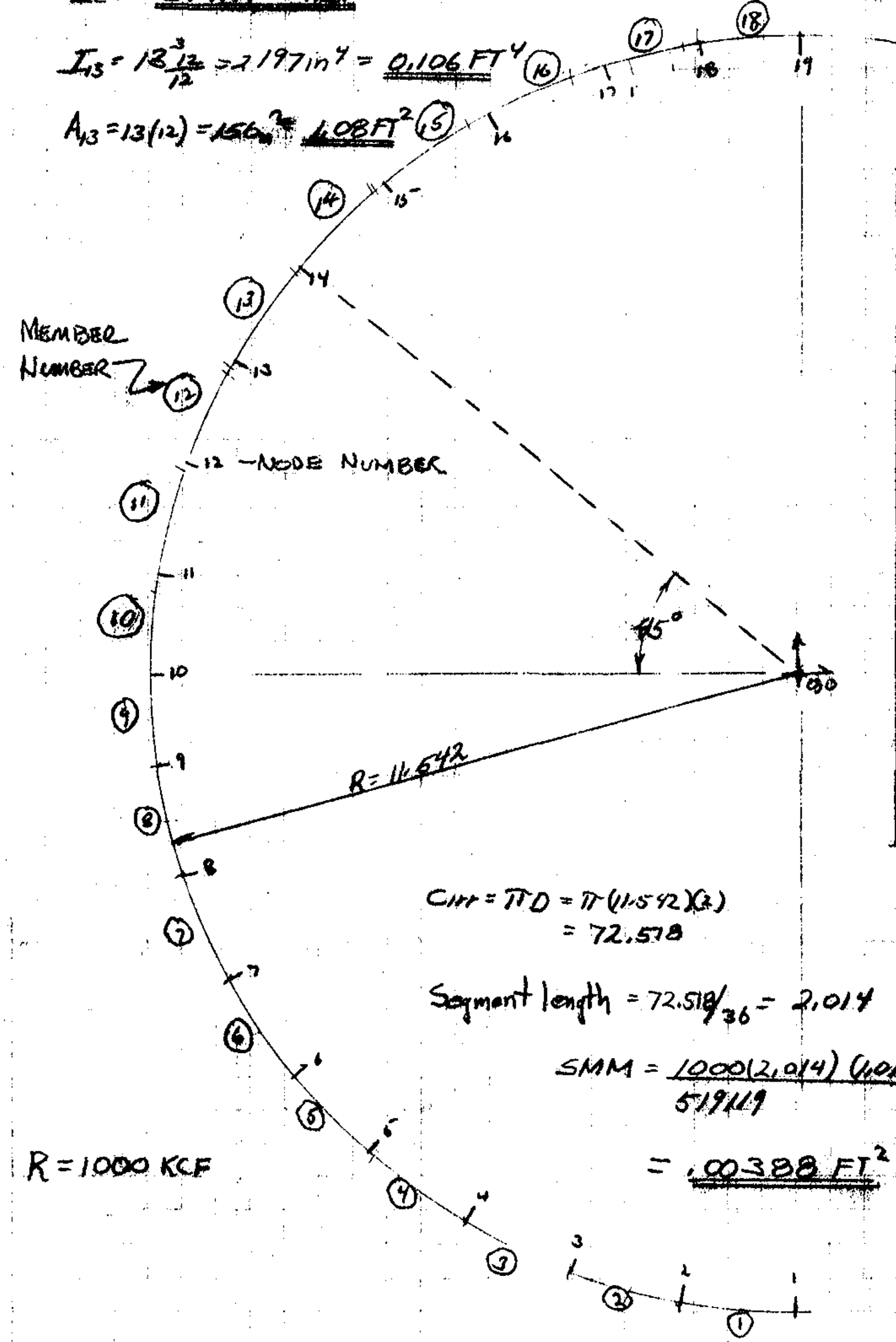
CONTENTS

SUBJECT Park River – Auxiliary Conduit Tunnel
COMPUTATION Computer Model – Cast-in-Place Drill And Blast Altern.
COMPUTED BY TKH CHECKED BY JF DATE Dec 76

Model for Sections I & II
Liner Concrete – $f_c' = 4000$ #/in² Design thickness = 13"
E_c = 519119 KSF

$I_{13} = \frac{13^3 (12)}{12} = 2197 \text{ in}^4 = 0.106 \text{ FT}^4$

$A_{13} = 13(12) = 156 \text{ in}^2 = 1.08 \text{ FT}^2$



Node	X	Y
1	0.0	-11.542
2	-2.004	-11.367
3	-3.948	-10.846
4	-5.771	-9.996
5	-7.419	-8.842
6	-8.842	-7.419
7	-9.996	-5.771
8	-10.846	-3.948
9	-11.367	-2.004
10	-11.542	0.0
11	-11.367	2.004
12	-10.846	3.948
13	-9.996	5.771
14	-8.842	7.419
15	-7.419	8.842
16	-5.771	9.996
17	-3.948	10.846
18	-2.004	11.367
19	0.0	11.542

$Circ = \pi D = \pi(11.542)(2) = 72.518$

Segment length $= 72.518/36 = 2.014$

$SMM = \frac{1000(2.014)(1.0)}{519119}$ (Support Modulus)

$= .00388 \text{ FT}^2$

R = 1000 KCF

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

SUBJECT Park River – Auxiliary Conduit Tunnel
COMPUTATION Computer Model C.I.P. – D&B
COMPUTED BY TKH CHECKED BY F DATE Dec 76

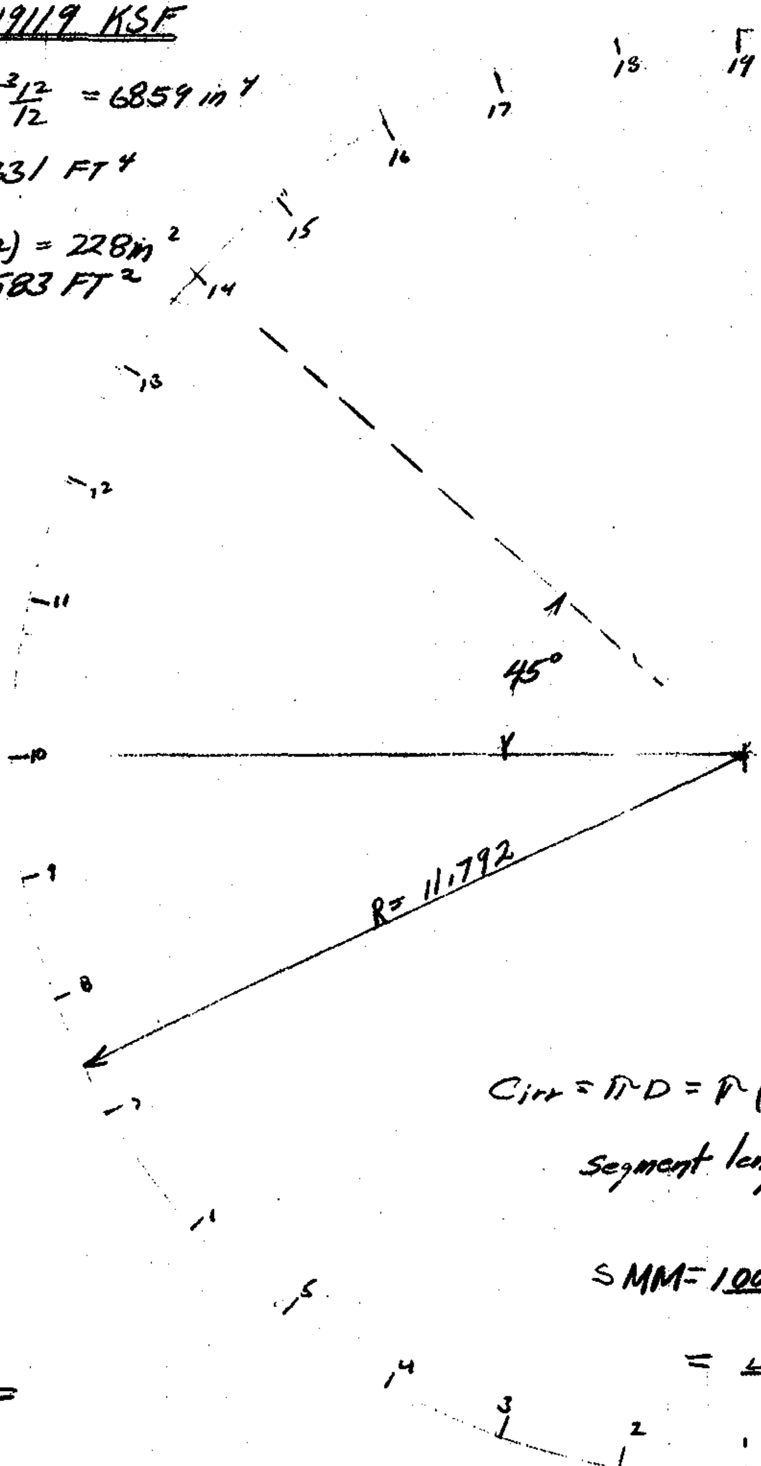
Model for Section III

$E_c = \underline{519119\ KSF}$

$I_{19} = \frac{19^3 \cdot 12}{12} = 6859\ in^4$

$= 0.331\ FT^4$

$A_{19} = 19(12) = 228\ in^2$
$= 1.583\ FT^2$



NODE	X	Y
1	0	-11.792
2	-2.048	-11.613
3	-4.033	-11.081
4	-5.896	-10.212
5	-7.580	-9.033
6	-9.033	-7.580
7	-10.212	-5.896
8	-11.081	-4.033
9	-11.613	-2.048
10	-11.792	0.0
11	-11.613	2.048
12	-11.081	4.033
13	-10.212	5.896
14	-9.033	7.580
15	-7.580	9.033
16	-5.896	10.212
17	-4.033	11.081
18	-2.048	11.613
19	0	11.792

$Cirr = \pi D = \pi (11.792)2 = 74.091$

Segment length $= 74.091/36 = 2.058$

$S\,MM = \frac{\underline{1000(2.058)10}}{519119}$
$= \underline{.00396\ FT^2}$

R = 1000 KCF

NED FORM 223
27 Sept. 49

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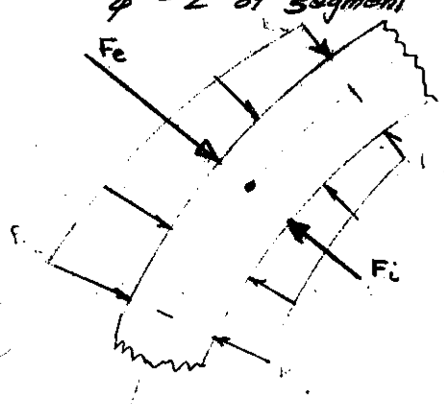
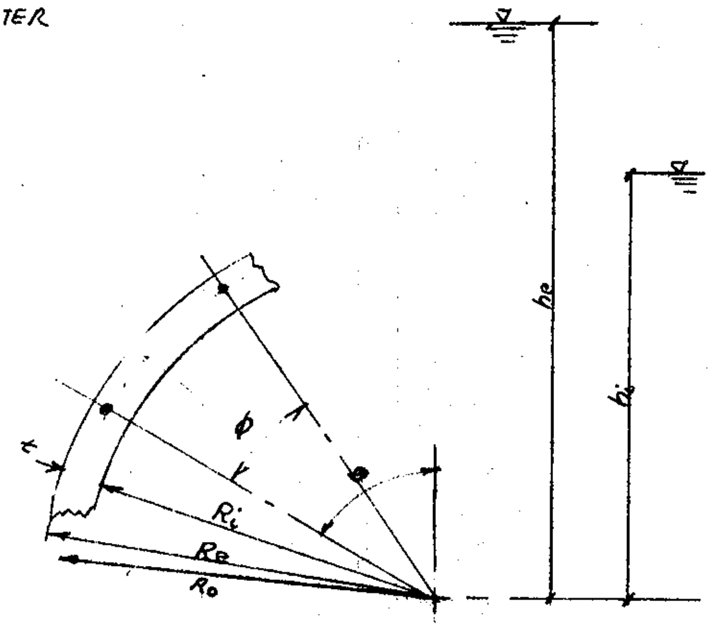
SUBJECT Park River – Auxiliary Conduit Tunnel
COMPUTATION Water Loads C.I.P. – D&B
COMPUTED BY TKH CHECKED BY JF DATE Dec 76

LC-1 EXTERNAL & INTERNAL WATER
LC-5 EXTERNAL WATER

h_i = Head internal water
h_e = Head external water
R_i = internal Radius
R_e = external Radius
θ = ∠ from vert
ϕ = ∠ of segment





F_i = Force due to internal head
F_e = Force due to external head
S_i = internal segment length
S_e = external segment length
F_n = net Force

$F_i = (h_i - \cos\theta\, R_i)\, S_i\, \gamma_w$
$F_e = (h_e - \cos\theta\, R_e)\, S_e\, \gamma_w$
$S_i = \pi 2 R_i / \#\text{of segments}$
$S_e = \pi 2 R_e / \#\text{of segments}$
$F_n = F_e - F_i$

for 13" liner 36 segment model $\phi = 10°$

$R_i = 11'$ $S_i = \pi 2 (11)/36 = 1.920$

$R_e = 12.083'$ $S_e = \pi 2 (12.083)/36 = 2.109$

LC-1 180 FT INTERNAL + 150 FT EXTERNAL HEAD

LC-1' 100 FT INTERNAL + 185 FT EXTERNAL HEAD

LC-5 185 EXTERNAL HEAD

NED FORM 223
27 Sept 49

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SUBJECT Park River - Auxiliary Conduit Tunnel
COMPUTATION Water Loads Sec I & II C.I.P. - D&B
COMPUTED BY TKH CHECKED BY JF DATE Dec 76

SEC I & II

Node	(−) INTERNAL 100 FT	(−) INTERNAL 180 FT	+ EXTERNAL 150 FT	+ EXTERNAL 185 FT	LC-1 180I/150E	LC-1' 100I/185E	LC5 185E
1	5.99	10.78	9.94	12.24	-0.84	6.25	12.24
2	11.96	21.54	19.86	24.46	-1.68	12.50	24.46
3	11.90	21.48	19.79	24.39	-1.69	12.49	24.39
4	11.80	21.39	19.67	24.28	-1.72	12.48	24.28
5	11.67	21.26	19.51	24.12	-1.75	12.45	24.12
6	11.51	21.09	19.31	23.92	-1.78	12.41	23.92
7	11.32	20.91	19.09	23.69	-1.82	12.37	23.69
8	11.11	20.70	18.84	23.44	-1.86	12.33	23.44
9	10.89	20.48	18.57	23.17	-1.91	12.28	23.17
10	10.66	20.25	18.29	22.90	-1.96	12.24	22.90
11	10.43	20.02	18.02	22.62	-2.00	12.19	22.62
12	10.21	19.80	17.75	22.35	-2.05	12.14	22.35
13	10.00	19.59	17.50	22.10	-2.09	12.10	22.10
14	9.82	19.40	17.27	21.88	-2.13	12.06	21.88
15	9.65	19.24	17.07	21.68	-2.17	12.03	21.68
16	9.52	19.11	16.92	21.52	-2.19	12.00	21.52
17	9.42	19.01	16.80	21.40	-2.21	11.98	21.40
18	9.36	18.95	16.73	21.33	-2.22	11.97	21.33
19	4.62	9.46	8.35	10.65	-1.11	6.03	10.65

WEIGHT OF 13" LINER

$$(\pi R_c^2 - \pi R_i^2)\,.150 = 11.78$$

per segment $11.78/30 = .32$

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27 Sept 49

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SUBJECT Park River - Auxiliary Conduit Tunnel
COMPUTATION Water Loads Sec I & II C.I.P D&B
COMPUTED BY TKH CHECKED BY F DATE Dec 76

SEC I & II

NODE	WEIGH Y	LC-1 Radial	LC-1 X	LC-1 Y	LC-1 Y+Weight	LC-1' Radial	LC-1' X	LC-1' Y	LC-1' Y+Weight	LC-5 Radial	LC-5 X	LC-5 Y	LC-5 Y+Weight
1	-.16	-0.84	0.0	-0.84	-1.00	6.25	0.0	6.25	6.09	12.24	0.0	12.24	12.08
2	-.32	-1.68	-0.29	-1.65	-1.97	12.50	2.17	12.31	11.99	24.46	4.25	24.09	23.77
3	-.32	-1.69	-0.58	-1.59	-1.91	12.49	4.27	11.74	11.42	24.39	8.34	22.92	22.60
4	-.32	-1.72	-0.86	-1.49	-1.81	12.48	6.24	10.81	10.49	24.28	12.14	21.03	20.71
5	-.32	-1.75	-1.12	-1.34	-1.66	12.45	8.00	9.54	9.22	24.12	15.50	18.48	18.16
6	-.32	-1.78	-1.36	-1.14	-1.46	12.41	9.51	7.98	7.66	23.92	18.32	15.38	15.06
7	-.32	-1.82	-1.58	-.91	-1.23	12.37	10.71	6.18	5.86	23.69	20.52	11.84	11.52
8	-.32	-1.86	-1.75	-.64	-.96	12.33	11.59	4.22	3.90	23.44	22.03	8.02	7.70
9	-.32	-1.91	-1.88	-.33	-.65	12.28	12.09	2.13	1.81	23.17	22.82	4.02	3.70
10	-.32	-1.96	-1.96	0.0	-.32	12.24	12.24	0.0	-0.32	22.90	22.90	0	-0.32
11	-.32	-2.00	-1.97	.35	.03	12.19	12.00	-2.12	-2.44	22.62	22.28	-3.93	-4.25
12	-.32	-2.05	-1.93	.70	.38	12.14	11.41	-4.15	-4.47	22.35	21.00	-7.64	-7.96
13	-.32	-2.09	-1.81	1.05	.73	12.10	10.48	-6.05	-6.37	22.10	19.14	-11.05	-11.37
14	-.32	-2.13	-1.63	1.37	1.05	12.06	9.24	-7.75	-8.07	21.88	16.76	-14.06	-14.38
15	-.32	-2.17	-1.39	1.66	1.34	12.03	7.73	-9.22	-9.54	21.68	13.94	-16.61	-16.93
16	-.32	-2.19	-1.10	1.90	1.58	12.00	6.00	-10.39	-10.71	21.52	10.76	-18.64	-18.96
17	-.32	-2.21	-.76	2.08	1.76	11.98	4.10	-11.26	-11.58	21.40	7.32	-20.11	-20.43
18	-.32	-2.22	-.39	2.19	1.87	11.97	2.08	-11.79	-12.11	21.33	3.70	-21.01	-21.33
19	-.16	-1.11	0.0	1.11	.95	6.03	0.0	-6.03	-6.19	10.65	0.0	-10.65	-10.81

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

PAGE F-6

SUBJECT Park River - Auxiliary Conduit Tunnel
COMPUTATION Water Loads Sec III C.I.P. D&B
COMPUTED BY JKH CHECKED BY J DATE Dec 76

for 19" liner 36 segment model $\phi = 10°$
$R_i = 11'$ $S_i = 1.920$
$R_e = 12.583'$ $S_e = 2.196$

LC-1, 180 FT INTERNAL + 150 FT EXTERNAL HEAD
LC-1' 100 FT INTERNAL + 185 FT EXTERNAL HEAD
LC-5 185 FT EXTERNAL HEAD

Node	(−) internal		(+) External		LC-1	LC-1'	LC-5
	100 FT	180 FT	150 F	185 F			
1	5.94	10.78	10.28	12.68	−.50	6.69	12.68
2	11.96	21.54	20.53	25.33	−1.01	13.37	25.33
3	11.90	21.48	20.46	25.26	−1.02	13.36	25.26
4	11.80	21.39	20.35	25.15	−1.04	13.35	25.15
5	11.67	21.26	20.20	25.00	−1.06	13.33	25.00
6	11.51	21.09	20.02	24.81	−1.07	13.30	24.81
7	11.32	20.91	19.80	24.60	−1.11	13.28	24.60
8	11.11	20.70	19.56	24.36	−1.14	13.25	24.36
9	10.89	20.48	19.31	24.11	−1.17	13.22	24.11
10	10.66	20.25	19.05	23.84	−1.20	13.18	23.84
11	10.43	20.02	18.79	23.58	−1.23	13.15	23.58
12	10.21	19.80	18.53	23.33	−1.27	13.12	23.33
13	10.00	19.59	18.29	23.09	−1.30	13.09	23.09
14	9.82	19.40	18.08	22.87	−1.32	13.05	22.87
15	9.65	19.24	17.89	22.69	−1.35	13.04	22.69
16	9.52	19.11	17.74	22.54	−1.37	13.02	22.54
17	9.42	19.01	17.63	22.43	−1.38	13.01	22.43
18	9.36	18.95	17.56	22.36	−1.39	13.00	22.36
19	4.62	9.46	8.77	11.17	−0.69	6.55	11.17

Weight of 19" Liner

$$(\pi R_e^2 - \pi R_i^2) .150 = 17.59$$

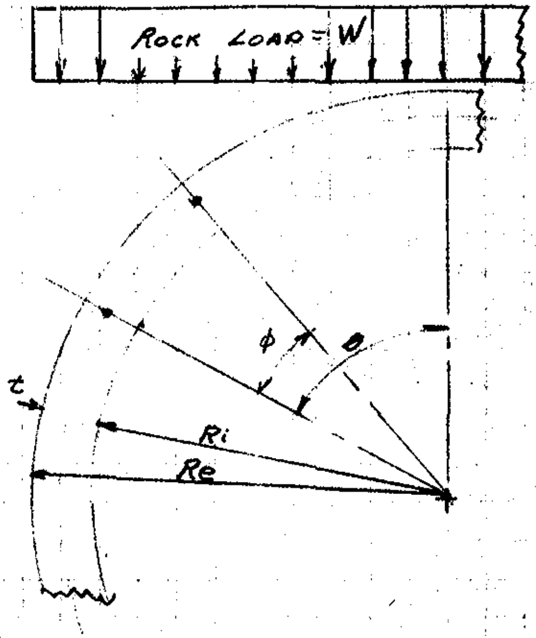
per segment $17.59/36 = .49$
SAY .50

SEC III

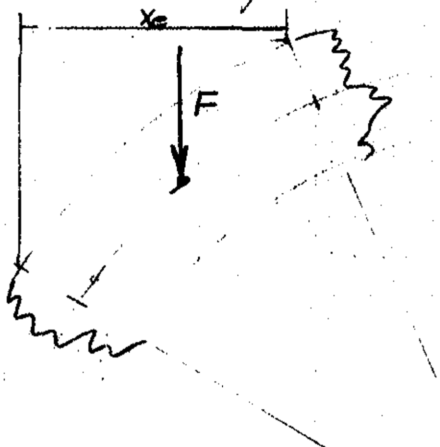
Node	Weight Y	LC-1 Radial	X	Y	Y+Weight	LC-2' Radial	X	Y	Y+Weight	LC-3 Radial	X	Y	Y+Weight
1	-,25	-,50	0	-,50	-,75	6,89	0	6,69	6,44	12,68	0	12,68	12,43
2	-,50	-1,01	-,18	-,99	-1,49	13,37	2,32	13,17	12,67	25,33	4,40	24,95	24,45
3		-1,02	-,35	-,96	-1,46	13,36	4,57	12,55	12,05	25,26	8,64	23,74	23,24
4		-1,04	-,52	-,90	-1,40	13,35	6,68	11,56	11,06	25,15	12,58	21,78	21,28
5		-1,06	-,68	-,81	-1,31	13,33	8,57	10,21	9,71	25,00	16,07	19,15	18,65
6		-1,07	-,82	-,69	-1,19	13,30	10,19	8,55	8,05	24,81	19,01	15,95	15,45
7		-1,11	-,96	-,56	-1,06	13,28	11,50	6,64	6,14	24,60	21,30	12,30	11,80
8		-1,14	-1,07	-,37	-,87	13,25	12,45	4,53	4,03	24,36	22,89	8,33	7,83
9		-1,17	-1,15	-,20	-,70	13,22	13,02	2,30	1,80	24,11	23,74	4,19	3,69
10		-1,20	-1,20	0,0	-,50	13,18	13,18	0,0	-,50	23,84	23,84	0,0	-,50
11		-1,23	-1,21	,21	-,29	13,15	12,95	-2,28	-2,78	23,58	23,22	-4,09	-4,59
12		-1,27	-1,19	,43	-,07	13,12	12,33	-4,49	-4,99	23,33	21,92	-7,98	-8,48
13		-1,30	-1,13	,65	,65	13,09	11,34	-6,54	-7,04	23,09	20,00	-11,54	-12,04
14		-1,32	-1,01	,85	,35	13,05	10,00	-8,39	-8,89	22,87	17,52	-14,70	-15,20
15		-1,35	-,87	1,03	,53	13,04	8,38	-9,99	-10,49	22,69	14,58	-17,38	-17,88
16		-1,37	-,68	1,19	,69	13,02	6,51	-11,28	-11,78	22,54	11,27	-19,52	-20,02
17		-1,38	-,47	1,30	,80	13,01	4,45	-12,23	-12,73	22,43	7,67	-21,08	-21,58
18	-,50	-1,39	-,24	1,37	,87	13,00	2,26	-12,80	-13,30	22,36	3,88	-22,02	-22,52
19	-,25	-0,69	0	,69	,44	6,55	0,0	-6,55	-7,05	11,17	0,0	-11,17	-11,42

SUBJECT PARK RIVER - AUXILIARY CONDUIT TUNNEL
COMPUTATION ROCK LOADS   C.I.P - D&B
COMPUTED BY TKH   CHECKED BY JY   DATE DEC 76

LC-2 ROCK LOAD



W = Rock LOAD T/FT
R_i = internal Radius
R_e = external Radius
θ = ∠ from vert
ϕ = ∠ of segment



X_e = horz projection of external face $X_e = R_e \sin\theta + \phi/2 - R_e \sin\theta - \phi/2$
F = Vertical Rock force $F = W X_e$

---

for 13" liner   36 segments $\phi = 10°$
$R_i = 11'$
$R_e = 12.083'$

LC-2   SECTION I   0.52 T/FT²
SECTION II   1.10 T/FT²
SECTION III   2.40 T/FT² *

LC-4   15 psi   Grout Pressure

$F = S_e(12)(12)(15) = 2.109(12)12(15) = \underline{4.56\ K}$

liner thickness 19"
$R_i = 11'$
$R_e = 12.583'$

SUBJECT Park River - Auxiliary Conduit Tunnel
COMPUTATION Rock Load C.I.P. - D&B
COMPUTED BY TKH CHECKED BY JF DATE Dec. 76

node	Xe		Weight		LC-2 Sec I		LC-2 Sec II		LC-2 Sec III	
	I&II	III	I&II	III	FY	Fy+Weight	Fy	Fy+Weight	Fy	Fy+Weight
1			.16	.25	0.	-.16	0	-.16	0	-.25
2			.32	.50	0	-.32	0	-.32	0	-.50
3					0	-.32	0	-.32	0	-.50
4					0	-.32	0	-.32	0	-.50
5					0	-.32	0	-.32	0	-.50
6					0	-.32	0	-.32	0	-.50
7					0	-.32	0	-.32	0	-.50
8					0	-.32	0	-.32	0	-.50
9					0	-.32	0	-.32	0	-.50
10	0.06	0.06			-.06	-.38	-.14	-.46	-.28	-.78
11	0.37	0.38			-.38	-.70	-.82	-1.14	-1.82	-2.32
12	0.72	0.75			-.74	-1.06	-1.58	-1.90	-3.60	-4.10
13	1.05	1.10			-1.10	-1.42	-2.32	-2.64	-5.28	-5.78
14	1.35	1.41			-1.40	-1.72	-2.98	-3.30	-6.76	-7.26
15	1.61	1.68			-1.68	-2.00	-3.54	-3.86	-8.06	-8.56
16	1.82	1.90			-1.90	-2.22	-4.00	-4.32	-9.12	-9.62
17	1.98	2.06			-2.06	-2.38	-4.36	-4.68	-9.88	-10.38
18	2.07	2.16	.32	.50	-2.16	-2.48	-4.56	-4.88	-10.36	-10.86
19	1.05	1.10	.16	.25	-1.10	-1.26	-2.64	-2.80	-5.28	-5.53

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Park River - Auxiliary Conduit Tunnel
COMPUTATION Loading Summary - Sec I C.I.P. D&B
COMPUTED BY TKH CHECKED BY JF DATE Dec. 76

Sec I

node	LC-1		LC-1'		LC-2		LC-3		LC-3'		LC-5		LC-5'	
	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y
1	0.0	-1.00	0.0	6.09	0	-.16	0.0	-1.00	0.0	6.09	0.0	12.08	0.0	12.08
2	-0.29	-1.97	2.17	11.99	0	-.32	-0.29	-1.97	2.17	11.99	4.25	23.77	4.25	23.77
3	-0.58	-1.91	4.27	11.42	0	-.32	-0.58	-1.91	4.27	11.42	8.34	22.60	8.34	22.60
4	-0.86	-1.81	6.24	10.49	0	-.32	-0.86	-1.81	6.24	10.49	12.14	20.71	12.14	20.71
5	-1.12	-1.66	800	9.22	0	-.32	-1.12	-1.66	8.00	9.22	15.50	18.16	15.50	18.16
6	-1.36	-1.46	9.51	7.66	0	-.32	-1.36	-1.46	9.51	7.66	18.32	15.06	18.32	15.06
7	-1.58	-1.23	10.71	5.86	0	-.32	-1.58	-1.23	10.71	5.86	20.52	11.52	20.52	11.52
8	-1.75	-0.96	11.59	3.90	0	-.32	-1.75	-0.96	11.59	3.90	22.03	7.70	22.03	7.70
9	-1.88	-0.65	12.09	1.81	0	-.32	-1.88	-0.65	12.09	1.81	22.82	3.70	22.82	3.70
10	-1.96	-0.32	12.24	-0.32	0	-.38	-1.96	-0.38	12.24	-0.38	22.90	-0.32	22.90	-0.38
11	-1.97	0.03	12.00	-2.44	0	-.70	-1.97	-0.35	12.00	-2.82	22.28	-4.25	22.28	-4.63
12	-1.93	0.38	11.41	-4.47	0	-1.06	-1.93	-0.36	11.41	-5.21	21.00	-7.96	21.00	-8.70
13	-1.81	0.73	10.48	-6.37	0	-1.42	-1.81	-0.37	10.48	-7.11	19.14	-11.36	19.14	-12.46
14	-1.63	1.05	9.24	-8.07	0	-1.72	-1.63	-0.35	9.24	-9.47	16.76	-14.38	16.76	-15.78
15	-1.39	1.34	7.73	-9.54	0	-2.00	-1.39	-0.34	7.73	-11.22	13.94	-16.93	13.94	-18.61
16	-1.10	1.58	6.00	-10.71	0	-2.22	-1.10	-0.32	6.00	-12.61	10.76	-18.96	10.76	-20.86
17	-0.76	1.76	4.10	-11.58	0	-2.38	-0.76	-0.30	4.10	-13.64	7.32	-20.43	7.32	-22.49
18	-0.39	1.87	2.08	-12.11	0	-2.48	-0.39	-0.29	2.08	-14.26	3.70	-21.33	3.70	-23.49
19	0.0	0.95	0.0	-6.19	0	-1.26	0.0	-0.16	0.0	-7.29	0.0	-10.81	0.0	-11.91

SEC II

node	LC-1		LC-1'		LC-2		LC-3		LC-3'		LC-5		LC-5'	
					X	Y								
1	SAME AS SEC I		SAME AS SEC I		0	-.16	0.0	-1.00	0.0	6.09	SAME AS SEC I		0.0	12.08
2					0	-.32	-0.29	-1.97	2.17	11.99			4.25	23.77
3					0	-.32	-0.58	-1.91	4.27	11.42			8.34	22.60
4					0	-.32	-0.86	-1.81	6.24	10.49			12.14	20.71
5					0	-.32	-1.12	-1.66	8.00	9.22			15.50	18.16
6					0	-.32	-1.36	-1.46	9.51	7.66			18.32	15.06
7					0	-.32	-1.58	-1.23	10.71	5.86			20.52	11.52
8					0	-.32	-1.75	-0.96	11.59	3.90			22.03	7.70
9					0	-.32	-1.88	-0.65	12.09	1.81			22.82	3.70
10					0	-.46	-1.96	-0.46	12.24	-0.46			22.90	-0.46
11					0	-1.14	-1.97	-0.79	12.00	-3.28			22.28	-5.07
12					0	-1.90	-1.93	-1.20	11.41	-6.05			21.00	-9.54
13					0	-2.64	-1.81	-1.59	10.48	-8.69			19.14	-13.68
14					0	-3.30	-1.63	-1.93	9.24	-11.05			16.76	-17.36
15					0	-3.86	-1.39	-2.20	7.73	-13.08			13.94	-20.47
16					0	-4.32	-1.10	-2.42	6.00	-14.71			10.76	-22.96
17					0	-4.68	-0.76	-2.60	4.10	-15.94			7.32	-24.79
18					0	-4.88	-0.39	-3.01	2.08	-16.67			3.70	-25.89
19					0	-2.80	0.0	-1.69	0.0	-8.83			0.0	-13.45

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Park River - Auxiliary Conduit Tunnel
COMPUTATION Loading Summary - Sec II C.I.P.-D&B
COMPUTED BY TCA CHECKED BY JF DATE Dec 76
PAGE F-11

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Park River – Auxiliary Conduit Tunnel
COMPUTATION Loading Summary – Sec III C.I.P. D/B
COMPUTED BY KH CHECKED BY JF DATE Dec 76

Sec III

node	LC-1		LC-1'		LC-2		LC-3		LC-3'		LC-5		LC-5'	
	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y
1	0	-.75	0	6.44	0	-.25	0	-.75	0	6.44	0	12.43	0	12.43
2	-0.18	-1.49	2.32	12.67	0	-.50	-0.18	-1.49	2.32	12.67	4.40	24.45	4.40	24.45
3	-0.35	-1.46	4.57	12.05	0	-.50	-0.35	-1.46	4.57	12.05	8.64	23.24	8.64	23.24
4	-0.52	-1.40	6.68	11.06	0	-.50	-0.52	-1.40	6.68	11.06	12.58	21.28	12.58	21.28
5	-0.68	-1.31	8.57	9.71	0	-.50	-0.68	-1.31	8.57	9.71	16.07	18.65	16.07	18.65
6	-0.82	-1.19	10.19	8.05	0	-.50	-0.82	-1.19	10.19	8.05	19.01	15.45	19.01	15.45
7	-0.96	-1.06	11.50	6.14	0	-.50	-0.96	-1.06	11.50	6.14	21.30	11.80	21.30	11.80
8	-1.07	-0.89	12.45	4.03	0	-.50	-1.07	-0.89	12.45	4.03	22.89	7.83	22.84	7.83
9	-1.15	-0.70	13.02	1.80	0	-.50	-1.15	-0.70	13.02	1.80	23.74	3.69	23.74	3.69
10	-1.20	-0.50	13.18	-.50	0	-.78	-1.20	-.78	13.18	-.78	23.84	-.50	23.84	-.78
11	-1.21	-0.29	12.95	-2.78	0	-2.32	-1.21	-2.11	12.85	-4.60	23.22	-4.59	23.22	-6.91
12	-1.19	-0.07	12.33	-4.99	0	-4.10	-1.19	-3.67	12.33	-8.59	21.12	-8.48	21.92	-12.58
13	-1.13	0.15	11.34	-7.04	0	-5.78	-1.13	-5.13	11.34	-12.32	20.00	-12.04	20.00	-17.82
14	-1.01	0.35	10.00	-8.89	0	-7.26	-1.03	-6.41	10.00	-15.65	17.52	-15.20	17.52	-21.96
15	-0.87	0.53	8.38	-10.49	0	-8.56	-0.87	-7.76	8.38	-18.55	14.58	-17.88	14.58	~~-25.79~~
16	-0.68	0.69	6.51	-11.78	0	-9.62	-0.68	-8.43	6.51	-20.90	11.27	-20.02	11.27	-29.14
17	-0.47	0.80	4.45	-12.73	0	-10.38	-0.47	-9.08	4.45	-22.61	7.67	-21.58	7.67	-31.46
18	-0.24	0.87	2.26	-13.30	0	-10.86	-0.24	-9.47	2.26	-23.66	3.88	-22.52	3.88	-32.88
19	0	0.44	0.0	-7.05	0	-15.53	0	-4.84	0.0	-12.33	0.0	-11.42	0.0	-16.70

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

PAGE F-13

SUBJECT Park River Auxiliary Conduit Tunnel
COMPUTATION Summary - Computed Results C.L.E. [illegible]
COMPUTED BY THH CHECKED BY [illegible] DATE Dec 1976

SECTION	LC	Max Pos Moments Node	Max Pos Moments M FT-KIPS	Max Pos Moments T KIPS	Max Neg Moment Node	Max Neg Moment M FT-KIPS	Max Neg Moment T KIPS	Max Thrust Node	Max Thrust M FT-KIPS	Max Thrust T KIPS	Max Shear Node	Max Shear V KIPS	Max Shear T KIPS
I Nominal Rock Bolt Support 13" Cast in Place 0.52 T/FT²	1	2	0.02	-7.23	16	-0.02	-10.56	19	-0.01	-10.90	4	-.004	-7.39
	1'	19	2.38	69.70	11	-2.37	71.76	2	.70	73.50	15	.44	70.46
	2	19	8.25	8.17	13	-6.73	15.56	2	.74	18.74	16	2.05	11.32
	3	19	7.55	-3.84	14	-5.69	2.24	2	.62	6.68	17	1.82	-2.15
	3'	19	11.73	76.13	13	-10.12	83.50	2	1.21	88.62	16	2.67	79.52
	4												
	5	1	4.94	136.16	9	-5.14	136.19	2	4.56	136.70	11	1.04	135.40
	5'	18	10.97	130.61	12	-9.46	139.42	2	1.45	141.34	16	2.39	133.36
II Heavy Rock Bolt Support 13" Cast in Place 1.10 T/FT²	1	2	0.02	-7.23	16	-0.02	-10.56	19	-0.01	-10.90	4	-.004	-7.39
	1'	19	2.38	69.70	11	-2.37	71.76	2	.70	73.50	15	.44	70.46
	2	19	18.08	16.83	13	-13.11	31.20	2	1.44	35.16	17	4.16	20.21
	3	19	17.36	4.96	14	-12.56	17.14	2	1.33	23.31	17	4.08	8.51
	3'	19	21.77	84.25	13	-17.86	91.30	2	1.90	102.85	16	4.41	90.88
	4												
	5	1	4.94	136.16	9	-5.14	136.19	2	4.56	136.70	11	1.04	135.40
	5'	19	21.32	138.57	13	-17.55	153.64	2	2.11	157.11	16	4.72	145.09
III Steel Sets 19" Cast in Place 2.40 T/FT²	1	19	2.19	-6.22	12	-1.92	-3.78	19	2.19	-6.22	16	.47	-5.55
	1'	19	6.44	75.79	11	-5.77	79.38	2	2.27	81.85	15	1.05	77.23
	2	19	60.34	29.61	13	-49.10	64.92	2	9.52	69.67	16	13.03	44.82
	3	19	59.34	22.11	13	-48.43	57.42	2	9.31	62.26	16	12.80	37.24
	3'	19	65.47	103.39	13	-52.47	139.47	2	10.93	143.72	16	13.80	118.90
	4												
	5	1	4.76	140.57	10	-5.29	139.33	2	4.47	141.12	13	.98	137.64
	5'	19	13.63	151.65	12	-51.25	197.50	2	11.49	191.21	16	13.41	172.95

LC-1 F-14

SEC I - 0.52 T/FT
NOMINAL SUPPORT
K=1000

CIP - D#3

MEMBER FORCES (LOCAL COORDINATES)

MEMBER	SHEAR	MOMENT	THRUST
1	.00332	.02377	-7.22635
2	-.00328	.01717	-7.28294
3	-.00417	.00877	-7.39097
4	-.00010	.00357	-7.55141
5	.00153	.01164	-7.75140
6	-.00357	.00426	-8.00637
7	-.00058	.00310	-8.28227
8	-.00015	.00281	-8.58591
9	-.00333	-.00383	-8.90159
10	.00216	.00045	-9.22271
11	-.00343	-.00646	-9.53622
12	.00022	-.00502	-9.84013
13	-.00091	-.00735	-10.11390
14	-.00291	-.01370	-10.35792
15	-.00249	-.01872	-10.55166
16	.00021	-.01830	-10.72473
17	.00370	-.01084	-10.83759
18	.00209	-.00663	-10.89738

BLOCKING POINT THRUSTS

JOINT	THRUST
1	-.63195
2	-1.25954
3	-1.27739
4	-1.30531
5	-1.33605
6	-1.36839
7	-1.42321
8	-1.46995
9	-1.52291
10	-1.58217
11	-1.63138
12	-1.69155
13	-1.73857
14	-1.78201
15	-1.82343
16	-1.85355
17	-1.88134
18	-1.89509
19	-.94597

ND JOINT REACTIONS

OINT	X-FORCE	MOMENT	Y-FORCE
FIRST	7.19851	-.01709	1.00000
LAST	10.85652	-.00653	-.95000

LC-1' SEC I 0.52 T/FT²

JOINT DISPLACEMENTS (GLOBAL COORDINATES)

JOINT	X	ROTATION	Y	RADIAL
1	0.	0.	-.00030	-.00030

[illegible]	[illegible]	[illegible]	[illegible]		[illegible]
16	.00053	-.00020	-.00503	F-15	.00451
17	.00042	-.00015	-.00545		.00527
18	.00024	-.00008	-.00575		.00571
19	0.	0.	-.00536		.00535

MEMBER FORCES (LOCAL COORDINATES)

LC-1'

SEC I 0.52T/FT²
NOMINAL SUPPORT
K=1000

C.I.P. D&S

MEMBER	SHEAR	MOMENT	THRUST
1	-.00093	.70445	73.50050
2	.03031	.76545	73.44206
3	.02653	.31392	73.33015
4	.00344	.82535	73.15735
5	-.06055	.70400	72.97033
6	-.17654	.34352	72.74504
7	-.32273	-.30053	72.50765
8	-.47372	-1.25394	72.27535
9	-.41297	-2.08458	72.03708
10	-.14204	-2.37042	71.75433
11	.11435	-2.14027	71.44572
12	.28234	-1.57135	71.11200
13	.39279	-.73111	70.77517
14	.43514	.09458	70.45936
15	.41356	.92662	70.17349
16	.35453	1.53992	69.94537
17	.24279	2.12858	69.78627
18	.12699	2.38405	69.69937

BLOCKING POINT THRUSTS

JOINT	THRUST
1	6.39510
2	12.79133
3	12.78942
4	12.79551
5	12.79881
6	12.81383
7	12.80941
8	12.76295
9	12.53252
10	12.24000
11	12.24137
12	12.25073
13	12.26094
14	12.26555
15	12.27631
16	12.27513
17	12.28392
18	12.28721
19	6.19000

ND JOINT REACTIONS

OINT	X-FORCE	MOMENT	Y-FORCE
FIRST	-73.22017	-.70643	-6.09000
LAST	-69.42305	2.38405	6.19000

LC-2 SEC. I

15	-.00129	-.00056	-.00442	F-16	.00256
16	-.00053	-.00061	-.00543		.00439
17	-.00018	-.00051	-.00549		.00504
18	.00000	-.00029	-.00729		.00718
19	0.	0.	-.00759		.00759

MEMBER FORCES (LOCAL COORDINATES)

LC-2

Sec I 0.52 T/FT²

Nominal Support

K=1000

C.I.P.-D&B

MEMBER	SHEAR	MOMENT	THRUST
1	.00069	.73957	18.74417
2	.01397	.75778	18.68720
3	.02524	.82055	18.57373
4	.04400	.90907	18.40701
5	.05291	1.01554	18.19210
6	.03175	1.07941	17.93860
7	-.05308	.9725	17.56225
8	-.24362	.48234	17.38544
9	-.54235	-.60963	17.13839
10	-.89778	-2.41559	15.88327
11	-1.14914	-4.72346	16.37056
12	-.99451	-5.72904	15.55321
13	.19030	-5.34617	14.39415
14	1.47038	-3.33712	12.92527
15	2.05410	.74546	11.32747
16	1.97349	4.72505	9.85031
17	1.36098	7.45413	8.75117
18	.39113	8.25099	3.16552

BLOCKING POINT THRUSTS

JOINT	THRUST
1	1.52995
2	3.25311
3	3.23367
4	3.20643
5	3.18051
6	3.16965
7	3.18335
8	3.24299
9	3.31035
10	3.31332
11	3.15219
12	2.52749
13	1.43097
14	1.10559
15	1.53209
16	1.92259
17	2.23644
18	2.44235
19	1.10000

ND JOINT REACTIONS

OINT	X-FORCE	MOMENT	Y-FORCE
FIRST	-18.57276	-.73829	.15000
LAST	-8.10026	8.25099	1.10000

13	-.00214	-.00010	-.00257	-.00057
14	-.00181	-.00030	-.00281	.00042
15	-.00126	-.00047	-.00337	.00177
16	-.00057	-.00053	-.00420	.00331
17	-.00024	-.00045	-.00511	.00472
18	-.00004	-.00026	-.00581	.00571
19	0.	0.	-.00507	.00607

F-17

MEMBER FORCES (LOCAL COORDINATES)

LC-3

SEC I 0.52 T/FT²
NOMINAL SUPPORT
K~1000

C.I.P. - D∅B

MEMBER	SHEAR	MOMENT	THRUST
1	.00401	.52085	5.68027
2	.00757	.63531	6.52271
3	.01639	.66928	6.51137
4	.03423	.73814	6.34665
5	.04285	.82436	5.13008
6	.02155	.85772	5.87928
7	-.04027	.78672	5.50451
8	-.18589	.41260	5.31975
9	-.41835	-.42896	5.05763
10	-.68561	-1.81018	4.77268
11	-.91022	-3.54209	4.22335
12	-.87675	-5.40559	3.37735
13	-.14305	-5.59338	2.23653
14	1.15254	-3.37397	.82732
15	1.79747	.24232	-.71897
16	1.82007	3.90327	-2.15245
17	1.32585	6.57169	-3.24743
18	.48610	7.54953	-3.84224

BLOCKING POINT THRUSTS

JOINT	THRUST
1	.57715
2	1.15719
3	1.13551
4	1.10330
5	1.07855
6	1.06775
7	1.06281
8	1.09670
9	1.13713
10	1.12243
11	1.00775
12	.52380
13	-.24143
14	-1.02363
15	-.63302
16	-.27235
17	.02195
18	.21790
19	.15000

ND JOINT REACTIONS

OINT X-FORCE MOMENT Y-FORCE

14	-.00190	-.00054	-.00672	.00257
15	-.00102	-.00035	-.00801	.00548
16	-.00024	-.00033	-.00963	.00822
17	.00020	-.00072	-.01124	.01063
18	.00024	-.00040	-.01242	.01228
19	0.	0.	-.01286	.01286

F-18

MEMBER FORCES (LOCAL COORDINATES)

LC-3'

Sec I 0.52 T/FT²
Nominal Suppor
K=K

C.I.P. D¢

MEMBER	SHEAR	MOMENT	THRUST
1	-.00285	1.20629	85.52837
2	.03752	1.28180	86.55952
3	.05535	1.39415	86.45435
4	.08092	1.55695	86.23274
5	.08414	1.72627	86.05537
6	.02592	1.77842	85.81014
7	-.14157	1.49345	85.53915
8	-.51912	.44855	85.29658
9	-1.03785	-1.63911	85.11543
10	-1.52213	-4.90223	84.96527
11	-1.77271	-8.45999	84.56705
12	-.82144	-10.12224	83.79585
13	1.03427	-7.94082	82.85285
14	2.22317	-3.46685	81.24359
15	2.57057	1.90521	79.51985
16	2.47899	6.89252	77.95616
17	1.73525	10.38490	75.77203
18	.66942	11.73153	75.13271

BLOCKING POINT THRUSTS

JOINT	THRUST
1	7.53902
2	15.07337
3	15.05427
4	15.03516
5	15.01592
6	15.03595
7	15.10507
8	15.25794
9	15.38726
10	15.37819
11	14.94383
12	13.71896
13	12.83093
14	13.16545
15	13.56377
16	13.92050
17	14.21956
18	14.40458
19	7.29000

ND JOINT REACTIONS

OINT	X-FORCE	MOMENT	Y-FORCE
FIRST	-85.29783	-1.21204	-6.09000
LAST	-75.78404	11.73153	7.29001

11	.00035	-.00023	.00439	F-19	-.00049
12	.00083	-.00032	.00425		-.00058
13	.00121	-.00031	.00355		-.00073
14	.00141	-.00027	.00233		-.00074
15	.00141	-.00021	.00215		-.00074
16	.00122	-.00015	.00157		-.00075
17	.00090	-.00010	.00114		-.00077
18	.00047	-.00005	.00038		-.00078
19	0.	0.	.00079		-.00079

MEMBER FORCES (LOCAL COORDINATES)

MEMBER	SHEAR	MOMENT	THRUST
1	-.18851	4.56327	136.70100
2	-.47399	3.59927	136.70009
3	-.75292	2.08434	136.70021
4	-.92358	.22673	136.68763
5	-.97833	-1.74221	136.65291
6	-.88214	-3.51698	136.57520
7	-.51057	-4.74530	136.42154
8	-.19505	-5.13936	136.19172
9	.42963	-4.27559	135.85244
10	1.03693	-2.13968	135.40312
11	.95105	-.27560	134.90995
12	.59156	.91428	134.47653
13	.27994	1.47748	134.13108
14	.06705	1.61243	133.85746
15	-.03812	1.53574	133.65013
16	-.05645	1.42218	133.49633
17	-.05225	1.31701	133.39637
18	.00215	1.32133	133.34157

LC-5

Sec I 0.52 T/FT²

Nominal Support

K=1000

C.I.P. D/B

BLOCKING POINT THRUSTS

JOINT	THRUST
1	12.08000
2	24.14688
3	24.08950
4	24.00538
5	23.87458
6	23.71432
7	23.53085
8	23.33497
9	23.11534
10	22.99269
11	23.57351
12	23.82506
13	23.72904
14	23.56556
15	23.41645
16	23.30949
17	23.24550
18	23.22189
19	11.50209

ND JOINT REACTIONS

10	-.00094	.00040	-.00534	F-20	-.00094
11	-.00163	.00022	-.00578		-.00060
12	-.00192	-.00007	-.00622		.00032
13	-.00159	-.00041	-.00688		.00193
14	-.00104	-.00070	-.00794		.00430
15	-.00025	-.00055	-.00941		.00705
16	.00037	-.00036	-.01113		.00983
17	.00061	-.00059	-.01277		.01220
18	.00043	-.00038	-.01395		.01381
19	0.	0.	-.01437		.01437

MEMBER FORCES (LOCAL COORDINATES)

LC-5'

Sec I 0.52T/FT³

Nominal Support K=1000

C.I.P.-D#3

MEMBER	SHEAR	MOMENT	THRUST
1	-.00592	1.45235	141.33797
2	.05082	1.55462	141.27475
3	.07035	1.69613	141.15554
4	.09428	1.88580	140.93235
5	.07194	2.03058	140.76719
6	-.04057	1.94395	140.52401
7	-.30094	1.34364	140.25956
8	-.73703	-.24033	140.05436
9	-1.36143	-2.97912	139.93193
10	-1.81376	-6.53779	139.82324
11	-1.40570	-9.46691	139.41607
12	.10379	-9.25813	138.53345
13	1.34107	-5.56008	137.13151
14	2.12960	-2.27442	135.55313
15	2.39346	2.54091	133.37373
16	2.15890	6.33335	132.35955
17	1.43050	9.86304	131.22459
18	.54957	10.96879	130.61122

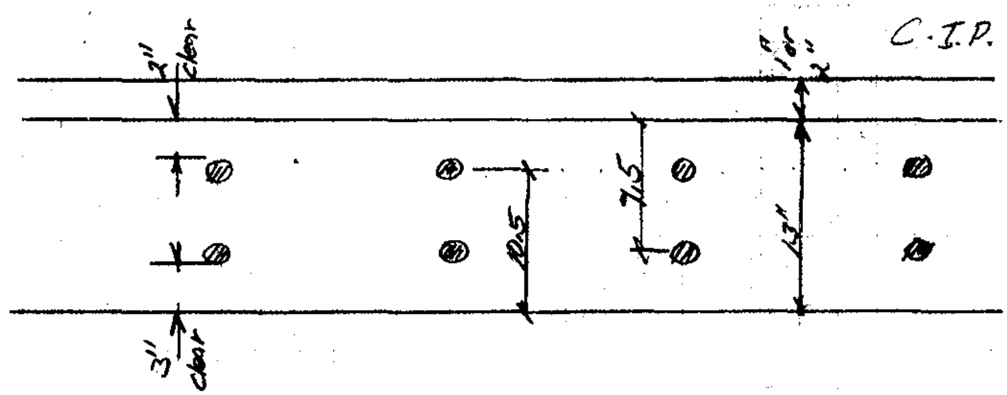
BLOCKING POINT THRUSTS

JOINT	THRUST
1	12.30249
2	24.50383
3	24.53334
4	24.57455
5	24.57505
6	24.62492
7	24.74042
8	24.90432
9	25.00532
10	24.79309
11	23.95593
12	22.70915
13	22.80571
14	22.98209
15	23.21655
16	23.44531
17	23.63725
18	23.77567
19	11.91000

ND JOINT REACTIONS

SUBJECT Park River - Auxiliary Conduit Tunnel
COMPUTATION Cast in Place Liner 13" Sections I & II
COMPUTED BY TKH CHECKED BY JR DATE Dec 76

C.I.P. -D&B



Positive Moments	Negative Moments
d = 9.5" d" = 3.0" d' = 2.5"	d = 10.5" d" = 4.0" d' = 3.5

.52 T/FT² Rock Load

LC	M	N	LC	M	N
3	7.55	-3.84	3	-5.69	2.24
3'	11.73	76.13	3'	-10.12	83.80
5'	10.97	130.61	5'	-9.46	139.43

1.10 T/FT Rock Load

LC	M	N	LC	M	N
3	17.36	4.96	3	-12.56	17.14
3'	21.77	84.25	3'	-17.86	94.30
5'	21.32	138.57	5'	-17.55	153.64

C.I.P. - D&B

LC-3 node 19 $M = 7.55^{'K}$ $N = -3.84^{K}$

Avg Compression

$$f_{c\,avg} = \frac{-3.84(1000)}{13(12)} = -24.62 \text{ psi}$$

24.62 psi Tension

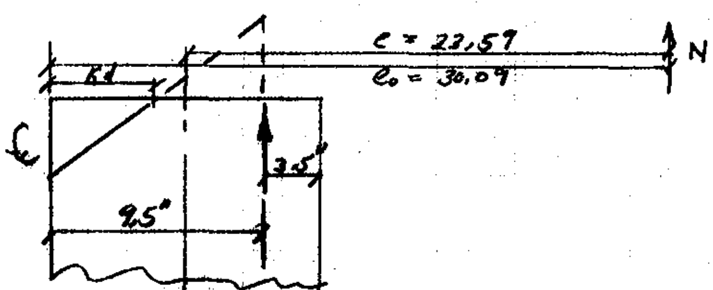
Max & Min Compression

$$f_c = \frac{-3.84(1000)}{13(12)} \pm \frac{7.55(12000)}{338} = -24.62 \pm 268.05$$

$= 292.67$ psi Tension
243.43 psi Compression

$e = \frac{12M}{N} = \frac{12(7.55)}{-3.84}$ $e_0 = -23.59 - 13/2$

$= -23.59$ $= -30.09$



Try $A_s = .60\,in^2$ $A_s' = 0.0$ (#7@12)

$\Sigma M_N = 0$

$$-\tfrac{1}{2} bkd\left(e_0 - Kd/3\right) + \left(\frac{d-Kd}{Kd}\right) 8 A_s \left(e_0 - d\right) = 0$$

$$-\tfrac{1}{2} 12 Kd\left(30.09 - Kd/3\right) + \left(\frac{9.5-Kd}{Kd}\right) 8(.60)\left(30.09 - 9.5\right) = 0$$

$$-180.54 Kd + 2Kd^2 + \frac{938.9 - 98.83Kd}{Kd} = 0$$

$$2Kd^3 - 180.54 Kd^2 - 98.83Kd + 938.9 = 0$$

Try $Kd = 2$ (35.08)
Try $Kd = 2.2$ (-131.04) Try $Kd = 2.04$ (2.93) Say $Kd = 2.05$ (65.19)

$$f_c = \frac{N}{\frac{bKd}{2} - \frac{d-Kd}{Kd}(n)A_s} = \frac{-3.84}{\frac{12(2.05)}{2} - \frac{9.5-2.05}{2.05}8(.6)} = \cancel{246.51 \text{ psi}}$$

SUBJECT Park River- Auxiliary Conduit Tunnel
COMPUTATION Cast in Place 13" Liner Sec I CIP-D&B
COMPUTED BY TKH CHECKED BY J DATE Dec 76

$$f_s = \frac{d-Kd}{Kd} 8 f_c = \frac{9.5-2.05}{2.05} 8(746.57) = \underline{21703.41 \text{ psi}}$$

> 20,000 psi no good

Try #8@12 $A_s = 0.79 \text{ in}^2$ less is required however keep 12" spacing to meet min steel #6@12" below 45° Line

$\Sigma M_N = 0$

$$-180.54Kd + 2Kd^2 + \frac{1236.22 - 130.13Kd}{Kd} = 0$$

$$2Kd^3 - 180.54Kd^2 - 130.13Kd + 1236.22 = 0$$

Try Kd = 2.2 (97.42)
Try Kd = 2.3 (6.20)
Try Kd = 2.35 (-40.66) Say Kd = 2.31 (-3.11)

$$f_c = \frac{-3.84}{\frac{12(2.31)}{2} - \left(\frac{9.5-2.31}{2.31}\right) 8(.79)} = \underline{660.78 \text{ psi}}$$

$$f_s = \frac{9.5-2.31}{2.31} 8(660.78) = \underline{16\,453.71} \text{ psi} < 20{,}000 \text{ psi ok}$$

<u>LC-3' node 17 M = 11.73'K N = 76.13 K</u>

<u>Avg Compression</u>

$$f_{c\,avg} = \frac{76.13(1000)}{156} = \underline{488.01 \text{ psi}}$$

<u>Max & Min Compression</u>

$$f_c = \frac{76.13(1000)}{156} \pm \frac{11.73(12000)}{338}$$

$$= 488.01 \pm 416.45$$

$$= \underline{904.46 \text{ psi}} \text{ Compression}$$
$$\underline{71.56 \text{ psi}} \text{ Compression}$$

SUBJECT Park River – Auxiliary Conduit Tunnel
COMPUTATION Cast in Place 13" Liner Sec I CIP.-D&B
COMPUTED BY TKH CHECKED BY J DATE Dec 16

LC-5' node 19 $M = 10.97'^k$ $N = 130.61^k$

Avg Compression

$$f_{c\,avg} = \frac{130.61(1000)}{156} = \underline{837.24\ psi}$$

Max & Min Compression

$$f_c = \frac{130.61(1000)}{156} \pm \frac{10.97(12000)}{338}$$

$$= 837.24 \pm 389.47$$

$= \underline{1226.71\ psi}$ Compression
$\underline{447.77\ psi}$ Compression

LC-3 node 14 $M = -5.69$ $N = 2.24^k$

Avg Compression

$$f_{c\,avg} = \frac{2.24(1000)}{156} = 14.36\ psi$$

Max & Min Compression

$$f_c = \frac{2.24(1000)}{156} \pm \frac{5.69(12000)}{338}$$

$$= 14.36 \pm 202.01$$

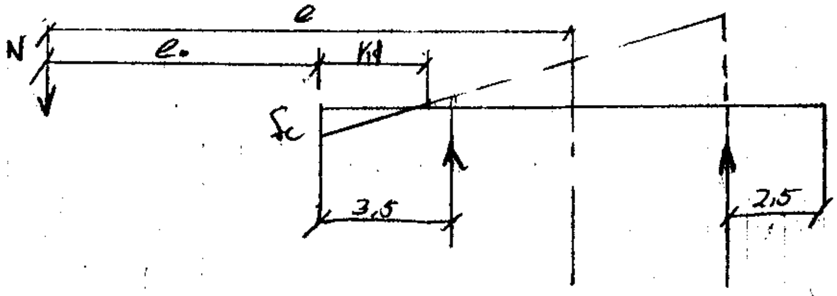
$= \underline{216.37\ psi}$ Compression
$= \underline{187.65\ psi}$ Tension

$e = 12M/N = \frac{12(5.69)}{2.24}$ $e_0 = 30.48 - 13/2$
$= 30.48$ $= 23.98$

Try $A_s = .44$ $A_s' = .44$
$\Sigma M_N = 0$
$$Kd^3 + 71.94Kd^2 + 151.37Kd - 954.58 = 0$$

Try $Kd = 3.5$ (499.36) Kd falls outside compression steel
re calc. $\Sigma M_N = 0$

SUBJECT Park River – Auxiliary Conduit Tunnel
COMPUTATION Cast in Place 13" Liner Sec I CIP. D&B
COMPUTED BY TKH CHECKED BY J DATE Dec 76



$\Sigma M_N = 0$

$$-\tfrac{1}{2} bkd\left(e_0 - Kd/3\right) + \frac{d' - Kd}{Kd}\, 8 A_s \left(e_0 + d'\right) + \frac{d - Kd}{Kd}\, 8 A_s \left(e_0 + d\right)$$

$$-\tfrac{1}{2}\, 12\, Kd\left(23.98 - Kd/3\right) + \frac{3.5 - Kd}{Kd}\, 8(.44)(23.98 + 3.5) + \frac{10.5 - Kd}{Kd}\, 8(.44)(23.98 + 10.5)$$

$$-6 Kd\left(23.98 - Kd/3\right) + \frac{3.5 - Kd}{Kd}(96.73) + \frac{10.5 - Kd}{Kd}(121.37)$$

$$2Kd^2 - 143.88Kd + \frac{338.56 - 96.73Kd}{Kd} + \frac{1274.39 - 121.37Kd}{Kd} = 0$$

$$2Kd^3 - 143.88Kd^2 - 218.10Kd + 1612.95 = 0$$

Try $Kd = 3.5$ (-827.18)
Try $Kd = 3$ (-282.27)
Try $Kd = 2$ (617.23) Try $Kd = 2.69$ (24.06) Say $Kd = 2.72$ (-4.52)

$$f_c = \frac{2.24}{\frac{12(2.72)}{2} - \left(\frac{3.5 - 2.72}{2.72}\right) 8(.44) - \frac{10.5 - 2.72}{2.72}\, 8(.44)} = \underline{\underline{427.29 \text{ psi}}}$$

$$f_{s_1} = \frac{3.5 - 2.72}{2.72}(8)(427.29) = \underline{\underline{980.25 \text{ psi}}}$$

$$f_{s_2} = \frac{10.5 - 2.72}{2.72}(8)(427.29) = \underline{\underline{9777.40 \text{ psi}}} < 20{,}000 \text{ psi OK}$$

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Park River - Auxiliary Conduit Tunnel
COMPUTATION Cast in Place 13" Liner Sec I CIP.D&B
COMPUTED BY JKH CHECKED BY ʃ DATE Dec 76

<u>LC-3' node 13 M = -10.12'K N = 83.80 K</u>

<u>Avg Compression</u>

$$f_{c\,avg} = \frac{83.80(1000)}{156} = \underline{\underline{537.18\text{ psi}}}$$

<u>MAX & MIN COMPRESSION</u>

$$f_c = \frac{83.80(1000)}{156} \pm \frac{10.12(12000)}{338}$$

= 896.47 psi Compression
<u>177.89 psi Compression</u>

<u>LC-5' node 12 M = -9.46 K N = 139.42 K</u>

<u>Avg Compression</u>

$$f_{c\,avg} = \frac{139.42(1000)}{156} = \underline{\underline{893.72\text{ psi}}}$$

<u>MAX & MIN Compression</u>

$$f_c = \frac{139.42(1000)}{156} \pm \frac{9.46(12000)}{338}$$

= 1229.58 psi Compression
<u>557.86 psi Compression</u>

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Park River — Auxiliary Conduit Tunnel
COMPUTATION Cast in Place 13" Liner Sec I CIP-D&B
COMPUTED BY TKH CHECKED BY [illegible] DATE Dec 76

Check Tension

LC-1 node 19 $M = -0.01$ $N = -10.90^{K}$
Assume full tension taken by inside ring of steel #8@12 @ node 19

$$f_s = \frac{10.90(1000)}{.79} = \underline{13,797.47\ psi} < 18000\ psi\ \text{ok}$$

Check Principal Stress

LC-3 node 17 $V = 1.82$ $N = -2.15$

$d = 9.5$
$A = 9.5(12) = 114$

$$v_c = \frac{1.82(1000)}{114} = 15.96\ psi \qquad f = \frac{-2.15}{114}(1000) = -18.86\ psi$$

$$f_t = \frac{-18.86}{2} \pm \sqrt{\left(\frac{-18.86}{2}\right)^2 + 15.96^2} = -9.43 \pm 18.54$$

9.11 psi Compression
27.97 psi Tension
allowable = 126 psi Tension ok

Check Bond Stress

Pos Steel
LC-3' node 15 $V = 2.67\ K$

#8 Σ 3.142

$$u = \frac{1000(2.67)}{3.142(7/8)9.5}$$

$= \underline{102.23\ psi}$

$u_{allow} = 304\ psi$

Neg Steel
LC-3' node 11 $V = -1.62\ K$

#6 Σ 2.356

$$u = \frac{1000(1.62)}{2.356(7/8)(9.5)}$$

$= \underline{74.89\ psi}$

$u_{allow} = 287\ psi$

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Park River – Auxiliary Conduit Tunnel
COMPUTATION Ring Beam
COMPUTED BY TJCH CHECKED BY DATE Dec 76

Model for Ring Beam

Rock Load 4.8 T/FT^2
Rib Spacing 2'-0" Blocking @ 48" max

Load per ft = 4.8(2)(2) = 19.2 K/FT
12" Over broke R=13.958

Segment 9°
Segment length = 1.977 ft.

Max blocking Spacing outside flang 48.85 in

Node	Horiz/proj	Vert load	Res Radial	X	Y
21	2.184	41.933	41.933	0	-41.933
19	4.153	79.738	68.36	21.12	-65.00
17	3.533	67.834	57.55	33.83	-46.56
15	2.567	49.285	47.12	38.12	-27.70
13	1.346	25.843	33.13	31.51	-10.24
11	.172	3.302	7.87	7.87	0
		267.936			

Support Modulus

E_s = 4176000 KSF

For Oak block E_b = 1.6 x 10^6 psi = 230400 KSF

A_b = 8 x 10 / 12 x 12
= .556

$$SMM_b = \frac{A_b}{L_b} \times \frac{E_b}{E_s} \times 1.0$$

$$= \frac{.556}{1} \times \frac{230400}{4176000} \times 1$$

$$= 0.0307 \ FT^2$$

19	.00019	-.00255	-.02594	[illegible]	
20	.00055	-.00173	-.03041	.03012	F-29
21	0.	0.	-.03252	.03252	

LC-2

W8x67 Ring Beam
4.8T/FT² Rock Load
Oak Blocking @ 48" max

MEMBER FORCES (LOCAL COORDINATES)

MEMBER	SHEAR	MOMENT	THRUST
1	-17.83396	-13.83160	209.93336
2	15.16989	16.12233	210.14692
3	-15.98473	-15.44755	210.21143
4	17.24330	18.50426	210.11154
5	-16.70734	-14.33310	210.06934
6	16.49455	18.13427	210.08667
7	-16.60593	-14.59934	210.09477
8	16.61548	18.21331	210.09395
9	-17.12674	-15.59962	210.13344
10	15.94907	15.83536	210.22612
11	-13.46623	-10.59835	210.03063
12	19.54346	27.90142	209.55223
13	-30.10522	-31.55659	210.37250
14	3.32856	-24.93530	212.43963
15	-13.45215	-51.54957	213.27924
16	20.21354	-11.63345	212.74500
17	-4.25572	-20.03515	211.49631
18	29.03361	37.30642	209.53774
19	-6.77332	23.93209	207.80545
20	25.81108	74.38563	205.30747

BLOCKING POINT THRUSTS

OINT	THRUST
1	34.31208
2	0.
3	63.94292
4	0.
5	66.69260
6	0.
7	65.34015
8	0.
9	66.51335
10	0.
11	62.32184
12	0.
13	82.35139
14	0.
15	50.00693
16	0.
17	57.55261
18	0.
19	68.34511
20	0.
21	41.93000

ND JOINT REACTIONS

OINT	X-FORCE	MOMENT	Y-FORCE
FIRST	-207.88395	-21.47301	-.00000

NED FORM 225
27 Sept 49

NEW ENGLAND DIVISION

CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Park River - Auxiliary Conduit Tunnel

COMPUTATION Ring Beam

COMPUTED BY TKH   CHECKED BY   DATE Dec 76

CRITICAL LOCATION member 20

$M = 74.89^{\prime k}$ $T = 206.31^{k}$

W8X67
$A = 19.7 in^2$
$d = 9.00 in$
$I = 272 in^4$
$S = 60.4 in^3$
$r = 3.71$

$$f_s = \frac{206.31(1000)}{19.7} + \frac{74.89(12)(1000)}{60.4}$$

$= 10472.59 + 14878.81$

$= 25351$ psi

* allowable stress temporary construction

*from EM 1110-2-2901

$F_y = 36{,}000$ psi

$f_a = .73 F_y = .73(36{,}000)$

$= 26{,}280$ psi > 25351 psi ok

USE W8X67

19	0.	0.	-.00124	.00124

MEMBER FORCES (LOCAL COORDINATES)

LC-1

SEC III 2.40 T/FT²
W/STEEL SETS K=1000

C.I.P-D¢B

MEMBER	SHEAR	MOMENT	THRUST
1	.00054	.45059	-.34459
2	.00335	.45743	-.42671
3	.00730	.47352	-.59391
4	.00578	.48541	-.85075
5	-.01221	.46031	-1.17254
6	-.05397	.33903	-1.55230
7	-.11423	.10426	-1.97677
8	-.18846	-.28303	-2.42234
9	-.26344	-.82461	-2.87431
10	-.29739	-1.43599	-3.32720
11	-.23620	-1.92140	-3.77306
12	.01551	-1.88951	-4.23342
13	.29260	-1.28800	-4.69714
14	.43339	-.39641	-5.14325
15	.46702	.56365	-5.54915
16	.41010	1.40669	-5.87060
17	.27849	1.97899	-6.09946
18	.10182	2.18832	-6.21333

BLOCKING POINT THRUSTS

JOINT	THRUST
1	-.03054
2	-.07008
3	-.09385
4	-.12421
5	-.15847
6	-.19100
7	-.25222
8	-.30960
9	-.38732
10	-.50614
11	-.68072
12	-.94926
13	-1.05360
14	-.99369
15	-.96522
16	-.93756
17	-.91252
18	-.89845
19	-.44000

END JOINT REACTIONS

JOINT	X-FORCE	MOMENT	Y-FORCE
FIRST	.34323	-.44948	.75000
LAST	6.20355	2.18832	-.44000

LC-1' SEC III 2.40 T/FT²

JOINT DISPLACEMENTS (GLOBAL COORDINATES)

JOINT	X	ROTATION	Y	RADIAL

F-32

12	------	-----		
13	.00003	-.00013	-.00342	.00174
14	.00013	-.00017	-.00376	.00256
15	.00030	-.00018	-.00416	.00338
16	.00035	-.00017	-.00457	.00413
17	.00031	-.00013	-.00493	.00474
18	.00018	-.00007	-.00513	.00513
19	0.	0.	-.00527	.00527

MEMBER FORCES (LOCAL COORDINATES)

LC-1'

SEC III 2.40T/FT²
W/STEEL SETS K=1000
C.I.P. D&B

MEMBER	SHEAR	MOMENT	THRUST
1	-.00534	2.26743	81.84894
2	.00233	2.27221	81.76404
3	-.02337	2.22416	81.59333
4	-.13417	1.94834	81.35147
5	-.29331	1.34563	81.05371
6	-.51174	.29365	80.74979
7	-.77790	-1.30550	80.42788
8	-.95242	-3.26277	80.10517
9	-.88274	-5.07751	79.77593
10	-.33682	-5.76994	79.38065
11	.23553	-5.28590	78.89847
12	.65378	-3.94193	78.34806
13	.90792	-2.07552	77.78353
14	1.04841	.07882	77.22320
15	1.04814	2.23346	76.72051
16	.91354	4.11145	76.29553
17	.70434	5.55936	75.97977
18	.45227	6.43977	75.79447

BLOCKING JOINT THRUSTS

JOINT	THRUST
1	7.13193
2	14.26379
3	14.26316
4	14.29749
5	14.31377
6	14.32536
7	14.29840
8	14.17013
9	13.87643
10	13.31398
11	13.23600
12	13.29313
13	13.34074
14	13.37432
15	13.42237
16	13.45677
17	13.48426
18	13.49039
19	7.05000

ND JOINT REACTIONS

OINT	X-FORCE	MOMENT	Y-FORCE
	[illegible]	[illegible]	[illegible]

LC-2

Sec III 2.40 T/FT²
W/STEEL SETS K=1000

C.I.P. D/B

MEMBER FORCES (LOCAL COORDINATES)

MEMBER	SHEAR	MOMENT	THRUST
1	.09737	9.52179	69.66374
2	.27953	10.09618	69.54339
3	.34533	10.80608	69.32221
4	.18584	11.18811	69.02439
5	-.28281	10.60694	68.71052
6	-1.13593	8.16702	68.45497
7	-2.59266	2.83725	68.35074
8	-4.35428	-6.11099	68.48700
9	-5.06772	-18.58505	68.90543
10	-6.92571	-32.82297	69.25821
11	-5.93114	-45.01171	68.09038
12	-1.99037	-49.10329	64.91651
13	5.98606	-36.79764	59.54267
14	11.10443	-13.97966	52.46411
15	13.03678	12.82013	44.82822
16	11.89762	37.27826	37.82053
17	8.26035	54.25363	32.49174
18	2.96258	60.34420	29.61617

BLOCKING POINT THRUSTS

JOINT	THRUST
1	5.96908
2	11.96199
3	12.04206
4	12.20432
5	12.47538
6	12.85940
7	13.31159
8	13.68529
9	13.69118
10	12.88464
11	10.98972
12	7.67054
13	2.89015
14	4.66657
15	6.55741
16	8.33102
17	9.75375
18	10.69501
19	5.53000

ND JOINT REACTIONS

OINT	X-FORCE	MOMENT	Y-FORCE
FIRST	-69.41159	-9.32161	.25000
LAST	-29.24473	60.34420	5.52998

LC-3 SEC III 2.4 T/FT²

JOINT DISPLACEMENTS (GLOBAL COORDINATES)

JOINT	X	ROTATION	Y	RADIAL

F-34

MEMBER FORCES (LOCAL COORDINATES)

LC-3

Sec III 2.40 T/FT^2
W/STEEL SETS K=1000

C.I.P. -D/B

MEMBER	SHEAR	MOMENT	THRUST
1	.09240	9.31243	62.26237
2	.26663	9.36044	62.14393
3	.33430	10.54369	61.92530
4	.18749	10.93406	61.62902
5	-.26430	10.39091	61.31323
6	-1.15150	8.02335	61.05135
7	-2.51711	2.84941	60.93249
8	-4.23902	-5.86196	61.05159
9	-5.92555	-18.04374	61.45032
10	-6.78723	-31.99697	61.77852
11	-5.87344	-44.06720	60.59022
12	-2.12299	-48.43142	57.41984
13	5.85343	-36.39343	52.06701
14	11.00460	-13.73555	44.99790
15	12.79594	12.51906	37.23906
16	11.70968	36.59079	30.27491
17	8.14136	53.32234	24.97608
18	2.92555	59.33726	22.11563

BLOCKING POINT THRUSTS

JOINT	THRUST
1	5.32915
2	10.67338
3	10.74972
4	10.90424
5	11.16879
6	11.55212
7	11.98115
8	12.35070
9	12.36550
10	11.58795
11	9.76354
12	6.55261
13	1.58654
14	3.33118
15	5.38535
16	6.96045
17	8.37140
18	9.30415
19	4.84000

ND JOINT REACTIONS

OINT	X-FORCE	MOMENT	Y-FORCE
FIRST	-62.03303	-9.12249	.75002
LAST	-21.77607	59.33726	4.83994

LC-3' SEC III 2.40 T/FT2

JOINT DISPLACEMENTS (GLOBAL COORDINATES)

JOINT	X	ROTATION	Y	RADIAL
1	0.	0.	-.00581	-.00581

10	[illegible]	[illegible]	[illegible]	[illegible]
11	-.00722	.00032	-.01328	-.00431
12	-.00745	-.00021	-.01358	-.00236
13	-.00664	-.00032	-.01435	.00142
14	-.00497	-.00137	-.01593	.00643
15	-.00295	-.00167	-.01840	.01220
16	-.00119	-.00167	-.02143	.01796
17	-.00012	-.00134	-.02439	.02288
18	.00019	-.00074	-.02656	.02619
19	0.	0.	-.02735	.02735

F-25

MEMBER FORCES (LOCAL COORDINATES)

LC-3'

SEC III -2.40 T/FT²
W/STEEL SETS K=1000

C.I.P. -D&B

MEMBER	SHEAR	MOMENT	THRUST
1	.09974	10.93242	143.72154
2	.30736	11.56402	143.60057
3	.37933	12.34384	143.36807
4	.16119	12.57521	143.06526
5	-.39999	11.85328	142.76073
6	-1.45442	8.86351	142.53397
7	-3.08736	2.51573	142.50141
8	-5.07375	-7.91101	142.74141
9	-6.93239	-22.16263	143.30300
10	-7.71034	-38.01469	143.79999
11	-6.14313	-50.53912	142.72270
12	-.88990	-52.46343	139.47010
13	6.96240	-38.15567	133.92134
14	11.98239	-13.53359	126.68095
15	13.30359	14.84237	118.89952
16	12.52750	40.59531	111.76709
17	8.76079	53.59924	106.33696
18	3.34074	65.46717	103.33763

BLOCKING POINT THRUSTS

JOINT	THRUST
1	12.41453
2	24.35531
3	24.94808
4	25.15617
5	25.47339
6	25.92431
7	26.44473
8	26.84766
9	26.80223
10	25.77367
11	23.43067
12	19.37008
13	15.98088
14	17.72001
15	19.59675
16	21.35479
17	22.76313
18	23.69299
19	12.33000

NO JOINT REACTIONS

15	.00095	-.00014	.00142	-.00045
16	.00035	-.00011	.00101	-.00045
17	.00063	-.00008	.00068	-.00043
18	.00033	-.00004	.00043	-.00041
19	0.	0.	.00041	-.00041

F36

MEMBER FORCES (LOCAL COORDINATES)

LC-5

SEC III 2.40T/FT
W/STEEL SETS K=1000
C.I.P. - D/B

MEMBER	SHEAR	MOMENT	THRUST
1	-.14269	4.46744	141.12541
2	-.37012	3.70635	141.03217
3	-.57097	2.53307	140.99567
4	-.75066	.98995	140.85630
5	-.82225	-.69967	140.66961
6	-.80816	-2.36102	140.42754
7	-.72701	-3.35554	140.12343
8	-.51105	-4.90530	139.76130
9	-.18543	-5.23702	139.33276
10	.19576	-4.88456	138.82992
11	.70314	-3.43953	138.26036
12	.97643	-1.43222	137.64262
13	.86118	.33303	137.05410
14	.64205	1.65740	136.53311
15	.46342	2.62032	136.03938
16	.24044	3.11460	135.72590
17	.10984	3.34033	135.51919
18	.04578	3.43444	135.41606

BLOCKING POINT THRUSTS

JOINT	THRUST
1	12.43000
2	24.84260
3	24.79352
4	24.71903
5	24.61632
6	24.49353
7	24.34633
8	24.18755
9	24.02010
10	23.84000
11	23.66428
12	23.73276
13	24.03133
14	24.07156
15	23.93775
16	23.88390
17	23.77903
18	23.69660
19	11.83634

END JOINT REACTIONS

JOINT	X-FORCE	MOMENT	Y-FORCE
FIRST	-140.57437	-4.76079	-12.43000
LAST	-134.89574	3.43444	11.42000

LC-1 SEC III

10	-.00554	.00067	-.01255	[illegible]
11	-.00661	.00029	-.01329	-.00420
12	-.00680	-.00025	-.01375	-.00169
13	-.00597	-.00086	-.01467	.00216
14	-.00433	-.00133	-.01640	.00722
15	-.00240	-.00167	-.01397	.01299
16	-.00077	-.00165	-.02206	.01872
17	.00016	-.00131	-.02503	.02358
18	.00033	-.00072	-.02719	.02683
19	0.	0.	-.02798	.02798

F-37

MEMBER FORCES (LOCAL COORDINATES)

LC-5'

Sec III 2.40T/FT²
W/STEEL SETS K=1000
C.I.P. - D&B

MEMBER	SHEAR	MOMENT	THRUST
1	.09570	11.49315	199.21268
2	.30581	12.12159	199.08922
3	.35860	12.35876	198.86237
4	.09782	13.05987	198.56768
5	-.51592	11.99970	198.27979
6	-1.64734	8.61330	198.08467
7	-3.36269	1.70062	198.08938
8	-5.33104	-9.25491	198.39231
9	-7.10965	-23.87100	198.99173
10	-7.64611	-39.58993	199.49864
11	-5.67509	-51.25254	197.88113
12	.14542	-50.95350	194.02667
13	7.19759	-36.15734	187.93243
14	11.81677	-11.87560	180.63565
15	13.41182	15.69502	172.95249
16	12.03377	40.44317	165.89911
17	8.30076	57.50165	[illegible]
18	2.98331	63.63471	157.[illegible]5635

BLOCKING POINT THRUSTS

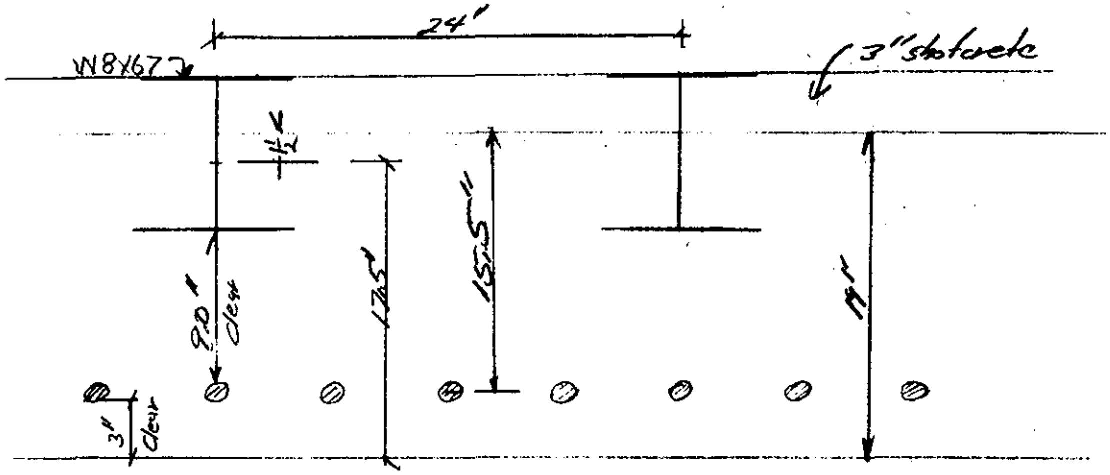
JOINT	THRUST
1	17.25017
2	34.53326
3	34.64342
4	34.86277
5	35.21074
6	35.63422
7	36.20245
8	36.52371
9	36.43432
10	35.23113
11	32.69363
12	28.37063
13	26.23067
14	27.53665
15	29.24310
16	30.87084
17	32.18576
18	33.05423
19	16.70000

ND JOINT REACTIONS

SUBJECT Park River – Auxiliary Conduit Tunnel

COMPUTATION Cast in Place 19" Liner w/ steel sets CIP.-D

COMPUTED BY TKH CHECKED BY J DATE Dec 7?



Assume full area of steel sets
sets with concrete

W8X67 $19.7\ in^2/24"$
Hole $1.305\ in^2$
$18.4\ in^2/24"$

AREA/FT

W8X67 $9.70\ in^2/FT$
#8@6" $1.58\ in^2/FT$ #9@6 $2.0\ in^2/FT$

POSITIVE MOMENTS

$d = 15.5$
$d'' = 6.0$
$d' = 1.5$

NEGATIVE MOMENTS

$d = 17.5$
$d'' = 8.0$
$d' = 3.5$

$2.4\ T/FT^2$

LC	M	N	LC	M	N
3	59.34	22.11	3	-48.43	57.42
3'	65.47	103.39	3'	-52.47	139.47
5'	63.63	157.65	5'	-51.25	197.50

SUBJECT Park River – Auxiliary Conduit Tunnel
COMPUTATION Cast in Place 19" Liner w/Steel Sets CIP-D&B
COMPUTED BY JKH CHECKED BY J DATE Dec 76

LC-3 node 19 $M = 59.34^{1K}$ $N = 22.11^{K}$

Avg Compression

$$f_{c\,avg} = \frac{22.11(1000)}{19(12)} = \underline{96.97\ psi}$$

Max & Min Compression

$$f_c = \frac{22.11(1000)}{19(12)} \pm \frac{59.34(12000)}{722}$$

$$= 1083.23 \text{ psi Compression}$$
$$\underline{889.27 \text{ psi Tension}}$$

$e = 12M/N = 12(59.34/22.11)$
$= 32.21$ in

$e_o = 32.21 - 19/2 = 22.71$ in   Try $A_s = 1.58\ in^2$ $A_s' = 9.70\ in^2$

$\Sigma M_N = 0$

$$Kd^3 + 68.13Kd^2 + 2002.76Kd - 6384.97 = 0$$

Try kd = 3 (263.48)
Try kd = 2.8 (~~+~~221.15)   Try kd = 2.89 (-3.83) ok

$$f_c = \frac{22.11}{\frac{12(2.89)}{2} + \frac{2.89-1.5}{2.89}16(9.70) - \frac{15.5-2.89}{2.89}8(1.58)} = 687.32 \text{ psi}$$

$$f_s = \frac{15.5-2.89}{2.89}8(687.32) = 25894.60 \text{ psi} > 20{,}000 \text{ psi}$$
no good.

Try #9@6 $A_s = 2.00\ in^2$ $A_s' = 9.70\ in^2$

$\Sigma M_N = 0$

$$Kd^3 + 68.13Kd^2 + 2066.96Kd - 7379.96 = 0$$

Try kd = ~~3~~ (-538.95)
Try kd = 3.5 (731.87)   Try 3.21 (-9.92)   Try 3.22 (16.44)

$$f_c = \frac{22.11}{\frac{12(3.21)}{2} + \frac{3.21-1.5}{3.21}15(9.7) - \frac{15.5-3.21}{3.21}8(2.0)} = \underline{622.35\ psi}$$

$$f_s = \frac{15.5-3.21}{3.21}8(622.35) = \underline{19062.13\ psi}$$
$< 20{,}000$ psi
ok

$$f_s' = \frac{3.21-1.5}{3.21}16(622.35)$$
$$= \underline{5304.52\ psi}$$

SUBJECT Park River – Auxiliary Conduit Tunnel
COMPUTATION Cast in Place 19" Liner w/ Steel Sets
COMPUTED BY TAH CHECKED BY J DATE Dec 76

CIP-D&B

LC-3' node 19 $M = 65.47^{1K}$ $N = 103.39^{K}$

Avg Compression

$$f_{c\,avg} = \frac{103.39(1000)}{19(12)} = \underline{453.46 \text{ psi}}$$

Max & Min Compression

$$f_c = \frac{103.39(1000)}{19(12} \pm \frac{65.47(12000)}{722}$$

$e = 12M/N = 12(65.47)/103.39$

$= 7.60$ in

$= 1541.60$ psi Compression
$\underline{634.68 \text{ psi Tension}}$

$e = 7.60 - 19/2 = -1.90$

$$f_{c\,max} = \frac{103.39(2)(1000)}{(12)(1.90)(3)} = 3023.10 \text{ psi} > 1800 \text{ psi}$$

no good
need reinf.

$A_s = 2.00$ $A_s' = 9.70$

$$Kd^3 - 5.70Kd^2 + 79.70Kd - 1642.75 = 0$$

Try kd = 6 (-1354.25)
Try kd = 10 (-415.75)
Try kd = 11 (-124.75)
Try kd = 12 (220.85) Try 11.36 (-6.94) Try 11.38 (-.18) o.k

$$f_c = \frac{103.39}{\frac{12(11.38)}{2} + \frac{11.38-1.5}{11.38}(15)9.7 - \frac{15.5-11.38}{11.38}8(2)} = \underline{547.59 \text{ psi}}$$

$$f_s = \frac{15.5-11.38}{11.38}(8)(547.59) = \underline{1585.99 \text{ psi}}$$

$$f_s' = \frac{11.38-1.5}{11.38}(16)(547.59) = \underline{7606.58 \text{ psi}}$$

SUBJECT Park River - Auxiliary Conduit Tunnel
COMPUTATION Cast in Place 19" Liner w/Steel Sets CIP-D&B
COMPUTED BY JKH CHECKED BY J DATE Dec 76

LC-5' node 19 $M = 63.68^{1K}$ $N = 157.65^{K}$

Avg Compression

$$f_{c\,avg} = \frac{157.65\,(1000)}{19(12)} = \underline{691.45\ psi}$$

Max & Min Compression

$$f_c = \frac{157.65(1000)}{19(12)} \pm \frac{63.63\,(12000)}{722}$$

$$= 1749.01\ psi \text{ Compression}$$
$$\underline{366.11\ psi} \text{ Tension}$$

$e = 12M/N = 12(63.63)/157.65$
$= 4.84$

$e_o = 4.84 - 19/2 = 4.66$

$$f_{c\,max} = \frac{157.65(2)(1000)}{12(4.66)(3)}$$

$$= \underline{1879.4\ psi} < 2400\ psi \text{ ok}$$

LC-3 node 13 $M = -48.43^{1K}$ $N = 57.42^{K}$

Avg Compression

$$f_{c\,avg} = \frac{57.42(1000)}{(19)(12)} = \underline{251.84\ psi}$$

Max & Min Compression

$$f_c = \frac{57.42(1000)}{(19)(12)} \pm \frac{48.43\,(12000)}{722}$$

$$= 1056.77\ psi \text{ Compression}$$
$$\underline{553.09\ psi \text{ Tension}}$$

$e = 12M/N = 12(48.43)/57.42$
$= 10.12$

$e_o = 10.12 - 19/2 = .62$

$\Sigma M_N = 0$

$$Kd^3 + 1.86\,Kd^2 + 764.86\,Kd - 12519.78\,Kd$$

Try $Kd = 16$ (4290.14)
Try $Kd = 10$ (-3685.18)

Try 12.77 (-366.76)
Try 13.00 (68.20)
Try 13.2 (200.43)

Say 13.05

SUBJECT PARK RIVER — AUXILIARY CONDUIT TUNNEL
COMPUTATION CAST IN PLACE 19" LINER w/ STEEL SETS CIP.-D&B
COMPUTED BY TKH CHECKED BY F DATE DEC 70

$$f_c = \frac{57.42}{\frac{12(13.05)}{2} + \frac{13.05-3.5}{13.05}(15)(2) - \frac{17.5-13.05}{13.05}8(9.7)} = \underline{\underline{778.13\ psi}}$$

$$f_s = \frac{17.5-13.05}{13.05}8(778.13) = \underline{\underline{2122.71\ psi}}$$

$$f_s' = \frac{13.05-3.5}{13.05}(16)(778.13) = \underline{\underline{9110.98\ psi}}$$

LC-3' node 13 $M = -52.47^{1K}$ $N = 139.47^K$

Avg Compression.

$$f_{c\,avg} = \frac{139.47(1000)}{19(12)} = 611.71\ psi$$

MAX & MIN COMPRESSION

$$f_c = \frac{139.47(1000)}{19(12)} \pm \frac{52.47(12000)}{722}$$

$e = 12M/N = 12(52.47)/139.47 = 4.51$

$= 1483.79$ psi Compression
$\underline{\underline{260.37\ psi}}$ Tension

$e_o = 4.51 - 19/2 = -4.99$

$$f_{c\,max} = \frac{139.47(2)(1000)}{12(4.99)(3)}$$

$$= \underline{\underline{1552.77\ psi}}$$

LC-5' node 13 $M = -51.125^{1K}$ $N = 197.50^K$

Avg Compression

$$f_{c\,avg} = \frac{197.50(1000)}{19(12)} = \underline{\underline{866.23\ psi}}$$

Max & Min Compression

$$f_c = 197.50(1000)/19(12) \pm \frac{51.25(12000)}{722}$$

1718.03 psi Compression
$\underline{\underline{14.43\ psi}}$ Compression

SUBJECT Park River – Auxiliary Conduit Tunnel

COMPUTATION Cast in Place 19" Liner w/ steel sets C.I.P. - D&B

COMPUTED BY TKH CHECKED BY J DATE Dec 76

Check Tension

LC-1 node 19 $M = 2.19^{\text{'K}}$ $N = -6.22^{K}$

Max fiber stress

$$f_c = \frac{-6.22(1000)}{19(12)} \pm \frac{2.19(12000)}{722}$$
$$= -27.28 \pm 36.40$$
$$= \underline{63.68 \text{ psi}}$$ tension < max

Tension in other LC in which steel was design also less than allowable

Check Principal Stress

LC-3 $V = 12.80$ $N = 37.24$

$d = 15.5$
$A = 15.5(12) = 186$

$$v_c = \frac{12.80(1000)}{186} = 68.82 \qquad f_c = \frac{37.24(1000)}{186} = 200.22$$

$$f_t = \frac{200.22}{2} \pm \sqrt{\left(\frac{200.22}{2}\right)^2 + (68.82)^2} = 100.11 \pm 121.48$$

= 221.59 psi Compression
21.37 psi Tension

allowable = 126 psi Tension

Check Bond Stress

Pos Steel
LC-3' node 16 $V = 13.80$
#9@6 (2)(3.544)

$$u = \frac{1000(13.80)}{2(3.544)\,7/8\;15.5}$$

$= \underline{143.55 \text{ psi}}$

allowable = 270 psi

Neg Steel
LC-3' node 11 $V = 7.71$
W8x67 Assum 1/2 of W8 is in Concrete $\Sigma_0 = 26.15$

$$u = \frac{1000(7.71)}{26.15\,(7/8)\,17.5}$$

$= \underline{19.25 \text{ psi}}$

allowable = $\frac{1}{2}$ max largest bar $B3/2$ = 66.50 psi

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

SUBJECT Park River - Auxiliary Conduit Tunnel
COMPUTATION Summary of Stress
COMPUTED BY TKH CHECKED BY JF DATE Dec 76
C.I.P.-D&B
PAGE F44

Section	Loading	Node	Moment M ft-kips	Thrust N kips	Avg Comp fc avg psi	Maximum Compression: Full Section fc max psi	Full Section fc min psi	Crack Section e in	Crack Section fc max psi	Transformed Area d in	eo in	Kd in	As in²	As' in²	fc psi	fs psi	fs' psi
Sec I 13" Liner Nominal Rock Support 0.52 T/FT²	3	19	7.55	-3.84	-25	2.43	-2.98	23.59		9.5	-30.09	2.31	0.79		661	16454	
	3'	19	11.73	76.13	488	904	72										
	5'	19	10.97	130.61	837	1227	448										
	3	14	-5.69	2.24	14	216	-188	30.48		10.5	23.98	2.69	0.44	0.44	427	9777	'980'
	3'	13	-10.12	83.80	537	896	178										
	5'	12	-9.46	139.42	894	1230	558										
Sec II 13" Liner Heavy Rock Support 1.1 T/FT²	3	19	17.36	4.96	32	648	-555	42.00		9.5	35.5	3.52	1.27		1285	17466	
	3'	19	21.77	84.25	540	1313	-233	3.10	1377								
	5'	19	21.32	138.57	888	1645	131										
	3	14	-12.56	17.14	110	556	-336	8.80		10.5	23.0	3.72	0.44	0.44	1052	15337	1182
	3'	13	-17.86	99.30	636	1271	2										
	5'	13	-17.55	153.64	985	1608	362										
Sec III 14" Liner Steel Support 2.4 T/ft²	3	19	59.34	22.11	97	1083	-889	32.21		15.5	22.71	3.21	2.0	9.7	622	19062	5305
	3'	19	65.47	103.39	453	1542	-635	7.6	3023	15.5	-1.90	11.38	2.0	9.7	548	1586	7606
	5'	19	63.63	157.65	691	1749	-366	4.84	1879								
	3	13	-48.43	57.42	252	1057	-553	10.12		17.5	0.62	13.05	9.7	2.0	778	2123	9111
	3'	13	-52.47	139.47	612	1484	-260	4.51	1553								
	5'	13	-51.25	197.50	866	1718	74										
Allowable					1000	1800	-100		1800						1800	20 000	16 000
" LC-5' only					1333	2400	-133		2400						2400	26666	21 333

' ' Tension

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

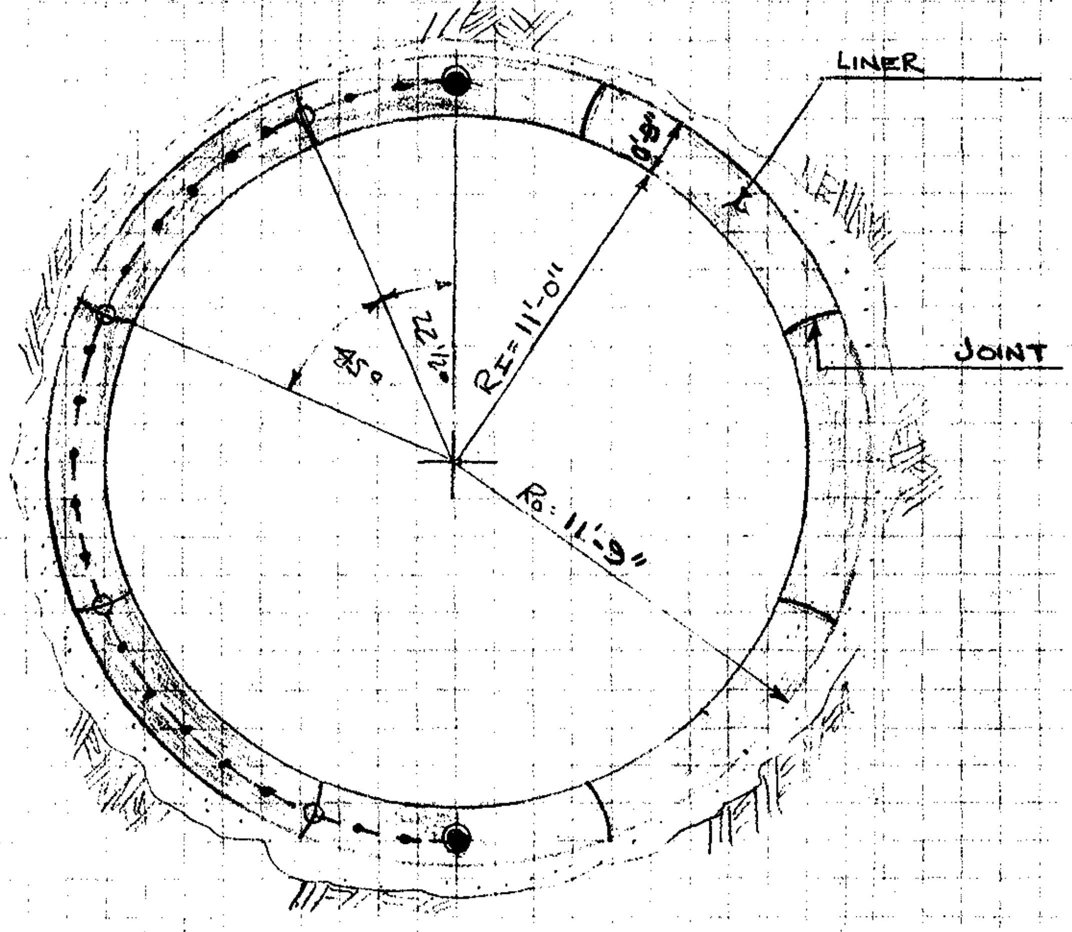
SUBJECT Park River - Auxiliary Conduit Tunnel
COMPUTATION Cast in Place Liners C.I.P. - D&B
COMPUTED BY JMH CHECKED BY DATE Dec 76

SUMMARY

SECTION	Positive Steel					Negative Steel					Other Stress			
	LC	AS	STEEL	LC	u / allow	LC	As	STEEL	LC	u / allow	LC	fs / allow	LC	fc / allow
SECTION I Nominal Support 0.52 T/FT²	3	0.79	#8@12	3'	102 psi / 304 psi	3	.44	#6@12	3'	75psi / 287psi	1	13797 psi / 20000 psi	3	28 psi / 126 psi
SECTION II Heavy Support 1.10 T/FT²	3	1.27	#10@12	3'	148 psi / 239 psi	3	.44	#6@12	3'	143 psi / 287 psi	1	13797 psi / 20000 psi	3	7 psi / 126 psi
SECTION III Steel Sets 2.40 T/FT²	3	2.0	#9@6	3'	144 psi / 270 psi	3	9.7	W8x67	3'	19psi / 666psi	1	not critical	3	21 psi / 126 psi

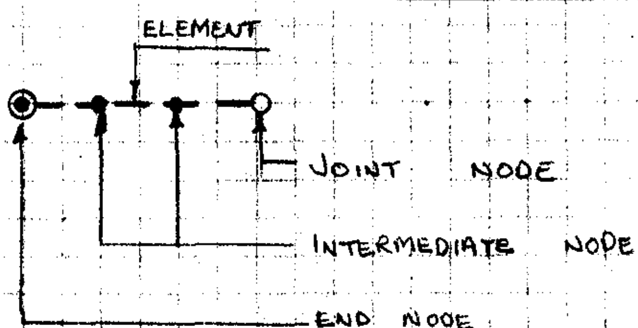
SUBJECT Park River
COMPUTATION Precast Tunnel
COMPUTED BY JF CHECKED BY TKH DATE Nov. '76

PRECAST TUNNEL



SECTION
N.T.S.

COMPUTER MODEL



NOTE: JOINT ORIENTATION IS STAGGERED 22½°. SECTION SHOWN IS JOINT POSITION 4 OF ANALYSIS.

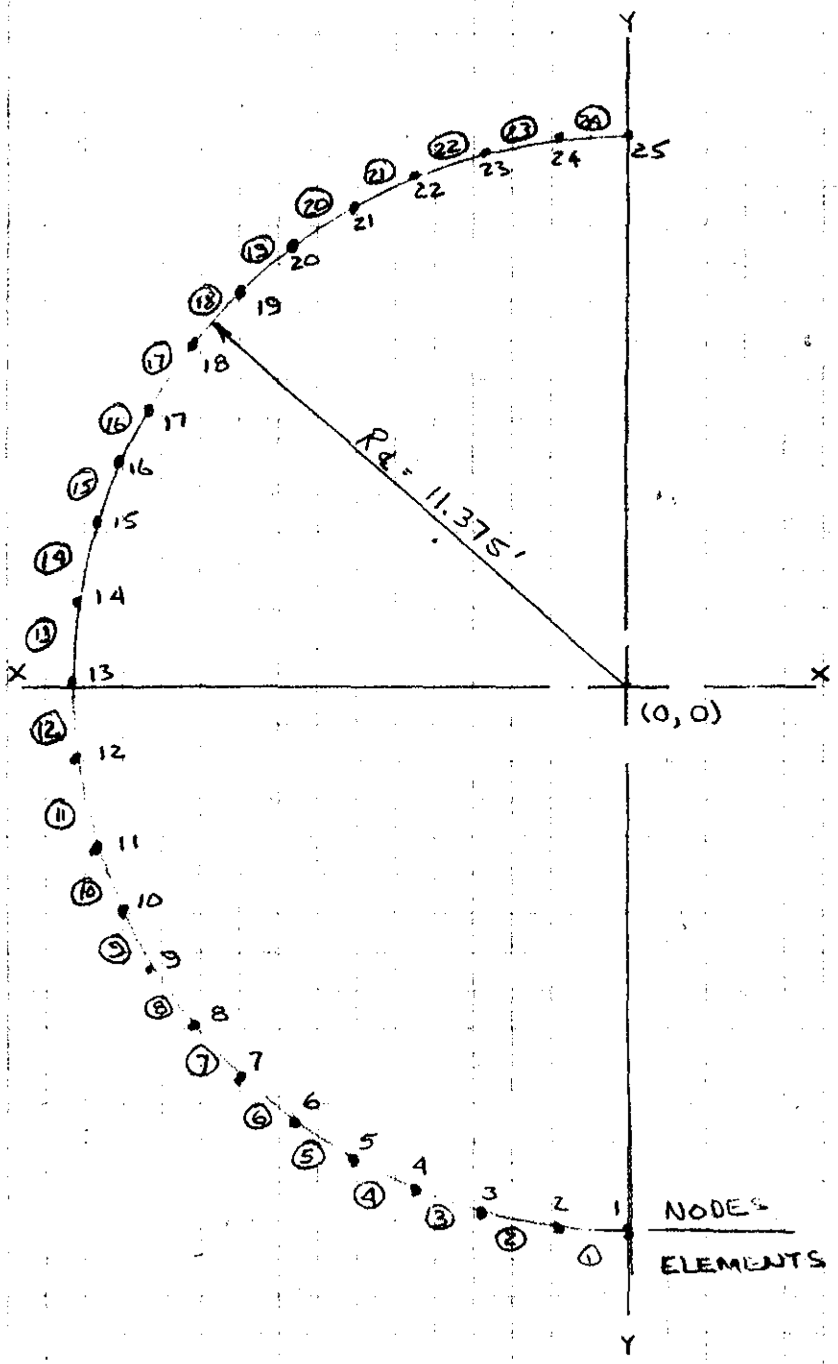
NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

PAGE F47

SUBJECT: PARK RIVER
COMPUTATION: PRECAST TUNNEL
COMPUTED BY: JF   CHECKED BY: D. D.   DATE: OCT. '76

PRECAST TUNNEL
24 Element Model
9' LINER



NODE	X	Y
1	0.00	-11.38
2	-1.48	-11.28
3	-2.94	-10.99
4	-4.35	-10.51
5	-5.69	-9.85
6	-6.92	-9.02
7	-8.04	-8.04
8	-9.02	-6.92
9	-9.85	-5.69
10	-10.51	-4.35
11	-10.99	-2.94
12	-11.28	-1.48
13	-11.38	0.00
14	-11.28	1.48
15	-10.99	2.94
16	-10.51	4.35
17	-9.85	5.69
18	-9.02	6.92
19	-8.04	8.04
20	-6.92	9.02
21	-5.69	9.85
22	-4.35	10.51
23	-2.94	10.99
24	-1.48	11.28
25	0.00	11.38

$t = 9''$

$I = \frac{1}{12}(1)\left(\frac{9}{12}\right)^3$

$I = 0.0352 \text{ FT.}^4$

$A = 0.75 \text{ FT.}^2$

$E = 552132 \text{ KSF}$

COMPUTER MODEL

TUNNEL PROGRAM

$$SMM = \frac{R}{E_s} A_s \times 1.0$$

$A_s = R_o \phi$

$A_s = 11.75 \times \frac{7.5 \times \pi}{180}$

$A_s = 1.539'$

SMM ≡ SUPPORT MODULUS
R ≡ COEFFICIENT OF SUBGRADE REACTION
E_s ≡ YOUNGS MODULUS FOR STRUCTURE
A_s ≡ CONTACT AREA FOR THE SEGMENT OF THE LINER CONSIDERED TO BE AN INDIVIDUAL ELEMENT.

R (KCF)	½ SMM	SMM	
500.0	0.00071	0.00142	
1000.0	0.00142	0.00284	← USE
2000.0	0.00284	0.00568	
3000.0	0.00426	0.00851	

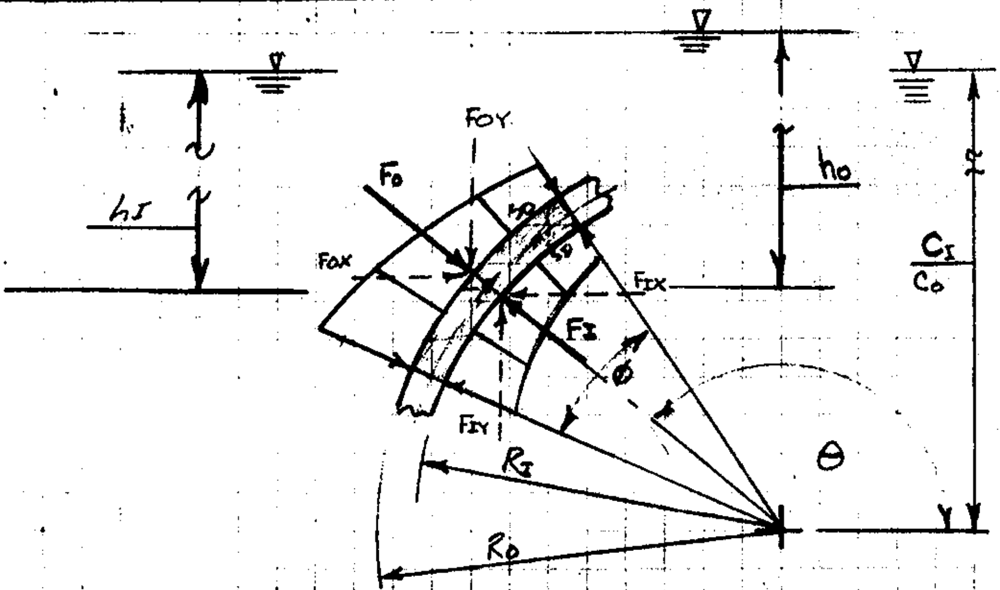
NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT PARK RIVER
COMPUTATION TUNNEL - LOADS 8
COMPUTED BY VF CHECKED BY TKH DATE 10/12/76

LC-11: EXTERNAL AND INTERNAL WATER LOAD

$S_o = R_o\ \phi$
$S_I = R_I\ \phi$



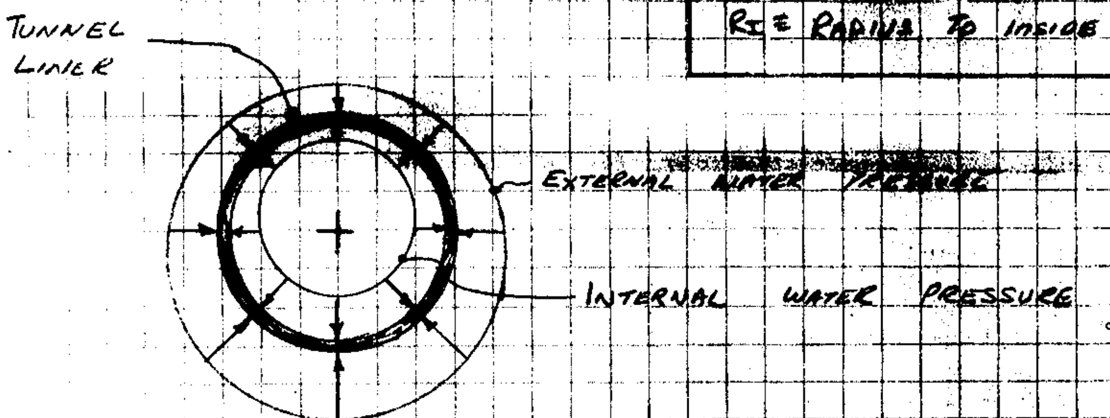
$F_o = \gamma_{WAT}\ h_o\ (S_o)$
$F_I = \gamma_{WAT}\ h_I\ (S_I)$

$F_{OX} = F_o\ \cos\theta$
$F_{OY} = F_o\ \sin\theta$
$F_{IX} = F_I\ \cos\theta$
$F_{IY} = F_I\ \sin\theta$

$F_N = F_o - F_I$
$F_{NX} = F_N\ \cos\theta$
$F_{NY} = F_N\ \sin\theta$

$h_I = C_I - R_I\ \sin\theta$
$h_o = C_o - R_o\ \sin\theta$

$C_I \equiv$ head of water inside tunnel at ℄
$C_o \equiv$ head of water outside tunnel @ ℄
$h_I \equiv$ head of water inside tunnel @ θ
$h_o \equiv$ head of water outside tunnel @ θ
$S_o \equiv$ outside length of tunnel, for ϕ
$S_I \equiv$ inside length of tunne, for ϕ
$R_o \equiv$ RADIUS TO OUTSIDE OF TUNNEL
$R_I \equiv$ RADIUS TO INSIDE OF TUNNEL



15	-5.40823
16	-5.42135
17	-5.44314
18	-5.46250
19	-5.47629
20	-5.49228
21	-5.51231
22	-5.52178
23	-5.50939
24	-5.52007
25	-2.76574

F50

PRECAST TUNNEL
9" LINER
JOINT POS. 4
K=1000 KCF
SECTION 1
LOADING CASES:
2, 3, 4' & 5'

12-6-76

ND JOINT REACTIONS

OINT	X-FORCE	MOMENT	Y-FORCE
FIRST	39.57733	-.03815	3.59999
LAST	42.16005	.03590	-3.53998

LC-2

SECTION 1

MEMBER FORCES (LOCAL COORDINATES)

MEMBER	SHEAR	MOMENT	THRUST
1	-.01676	.07549	11.31795
2	.01404	.09517	11.29562
3	-.06544	0.	11.25458
4	.02605	.03834	11.19241
5	.03022	.08282	11.10355
6	.00985	.09732	10.99721
7	-.01484	.07548	10.87707
8	-.03165	.02891	10.74496
9	-.01965	0.	10.50078
10	.08799	.12932	10.43853
11	.04031	.18859	10.26626
12	.08854	.31897	10.08853
13	.07057	.42281	9.85774
14	-.02258	.38950	9.57539
15	-.26474	0.	9.22740
16	-.71897	-1.05799	8.83818
17	-.73811	-2.14421	8.43035
18	-.14492	-2.35746	7.94757
19	.32295	-1.88221	7.39734
20	.59523	-1.00525	5.83084
21	.68381	0.	5.30105
22	.63247	.92954	5.85342
23	.39559	1.51229	5.53845
24	.14925	1.73193	5.37015

BLOCKING POINT THRUSTS

JOINT	THRUST
1	.75552
2	1.49375
3	1.46616
4	1.41924
5	1.45453
6	1.45519
7	1.45625

F51

12	1.32411
13	1.32139
14	1.35435
15	1.47184
16	1.63410
17	1.14888
18	.47379
19	.53740
20	.65349
21	.77077
22	.86915
23	.93691
24	.93128
25	.50000

ND JOINT REACTIONS

OINT	X-FORCE	MOMENT	Y-FORCE
FIRST	-11.29113	-.10015	.03000
LAST	-5.34791	1.73193	.49999

MEMBER FORCES (LOCAL COORDINATES)

LC-3

SECTION 1

MEMBER	SHEAR	MOMENT	THRUST
1	-.03020	.12324	76.79547
2	.24191	.43452	76.77220
3	-.32957	0.	76.73517
4	.07050	.10390	76.66523
5	.07663	.21667	76.56751
6	.02253	.24982	76.45318
7	-.03388	.19997	76.33347
8	-.08139	.08019	76.21424
9	-.05450	0.	76.07257
10	.39357	.57843	75.91355
11	-.10981	.41571	75.73650
12	.20324	.71579	75.54373
13	.13247	.91073	75.30132
14	-.07443	.80120	75.01411
15	-.54444	0.	74.63523
16	-1.34099	-1.97332	74.34931
17	-.57188	-2.81492	73.97506
18	.02472	-2.77855	73.47512
19	.45591	-2.10767	72.90507
20	.[illegible]8339	-1.10126	72.32353
21	.74837	0.	71.73758
22	.92560	1.36035	71.33740
23	.15936	1.59577	71.01635
24	.15389	1.82959	70.84640

BLOCKING POINT THRUSTS

JOINT	THRUST
1	5.05051
2	10.03122
3	10.01115
4	9.92339

F52

5	[illegible]
6	1.67545
7	1.68168
8	1.66829
9	1.53085
10	1.55667
11	1.60434
12	1.64553
13	1.69005
14	1.73313
15	1.93340
16	2.11763
17	2.09440
18	1.79993
19	.93506
20	0.
21	0.
22	0.
23	3.34252
24	3.33912
25	1.67000

ND JOINT REACTIONS

OINT	X-FORCE	MOMENT	Y-FORCE
FIRST	-12.53753	-.11517	.00000
LAST	-11.53375	11.93529	1.57000

MEMBER FORCES (LOCAL COORDINATES)

LC-4'

SECT. S 1-3

MEMBER	SHEAR	MOMENT	THRUST
1	-.03195	.15447	25.51946
2	.03109	.21025	25.51961
3	-.14306	0.	25.52630
4	.08955	.13179	25.53005
5	.11802	.30547	25.51545
6	.08217	.42638	25.50334
7	.01325	.44588	25.49708
8	-.08928	.31449	25.50207
9	-.21372	0.	25.52195
10	-.22649	-.33288	25.55149
11	-.24189	-.68913	25.58057
12	.46876	-.00555	25.55518
13	-.00057	-.00033	25.53455
14	.00094	.00106	25.53029
15	-.00071	0.	25.53568
16	-.00709	-.01043	25.53614
17	-.00357	-.01567	25.53557
18	-.00268	-.01962	25.53866
19	.00268	-.01566	25.53357
20	.00355	-.01042	25.53557
21	.00708	0.	25.53519
22	.09945	.14517	25.53554
23	-.10294	-.00542	25.53015
24	.00071	-.00435	25.53452

BLOCKING POINT THRUSTS

F53

4	3.20354
5	3.31127
6	3.37126
7	3.40661
8	3.43545
9	3.46237
10	3.44963
11	3.15997
12	2.73893
13	3.80899
14	3.33911
15	3.34252
16	3.34462
17	3.33781
18	3.33817
19	3.33754
20	3.33817
21	3.33781
22	3.34452
23	3.34252
24	3.33912
25	1.67000

ND JOINT REACTIONS

OINT	X-FORCE	MOMENT	Y-FORCE
FIRST	-25.45935	-.21149	.00000
LAST	-25.47658	-.00435	1.67000

MEMBER FORCES (LOCAL COORDINATES)

LC - 5'

SECTION 1

MEMBER	SHEAR	MOMENT	THRUST
1	-.05510	.51848	138.81814
2	.40400	1.11349	138.81395
3	-.75762	0.	138.79470
4	-.26770	-.39394	138.76440
5	-.24594	-.75587	138.71224
6	-.17995	-1.02066	138.63521
7	-.02157	-1.05241	138.52893
8	.19830	-.75985	138.38262
9	.51638	0.	138.18702
10	.78731	1.15712	137.97569
11	-.45756	.48324	137.78525
12	-.00932	.46880	137.61557
13	-.10421	.31545	137.40252
14	-.13330	.11885	137.13915
15	-.08076	0.	135.78311
16	.07419	.10917	135.33275
17	.10603	.25520	135.81870
18	.04773	.33545	135.27037
19	-.02502	.29715	134.73039
20	-.10701	.13959	134.23257
21	-.09492	0.	133.79630
22	.57971	.85200	133.43524
23	-.42053	.23257	133.17614
24	.06049	.32169	133.03509

[illegible]

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

SUBJECT PARK RIVER
COMPUTATION PRECAST TUNNEL
COMPUTED BY JF CHECKED BY D.D. DATE NOV. '76

PRECAST TUNNEL DESIGN

CONSTANTS:

$b = 12$ in.
$t = 9$ in.
$a = 1.44$
$f'_c = 5000$ psi

ITEM	+M	-M
d	7.5	5.5
d'	1.5	3.5
d''	3.5	1.5

$As_{min} = 0.0025\ bt$
$As_{min} = 0.0025 \times 12 \times 9$
$As_{min} = 0.27\ in^2$

$As_{temp} = 0.00125\ bt$
$As_{temp} = 0.00125 \times 12 \times 9$
$As_{temp} = 0.14\ in^2$

$n = 7.1$
$K = 426$
$j = 0.852$

	1.00	1.33 (LC-5')
f_c =	2250. psi	2993. psi
f_s =	20.0 ksi	26.7 ksi
f'_s =	20.0 ksi	26.7 ksi

LOAD CASES:

LC-1 185 FT. EXTERNAL HEAD, 100 FT. INTERNAL HEAD
LC-2 VERTICAL ROCK LOAD
LC-3 174 FT. EXTERNAL HEAD, 89 FT. INTERNAL HEAD + ROCK LOAD
LC-4 GROUT PRESSURE, $\theta = 37.5°$
LC-4' " " , $\theta' = 180°$
LC-5' 180 FT. EXTERNAL HEAD + ROCK LOAD (increase allowable stress by 33%)

ROCK LOADS

SECT.	ω TSF	ω KSF
1	0.25	0.54
2	0.70	1.50
3	1.75	3.75

REFERENCES:

1. EM 1110-2-2902 DESIGN OF CONDUITS, CULVERTS AND PIPES
2. EM 1110-2-2901 - DESIGN OF MISC. STRUCTURES - TUNNELS

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT PARK RIVER
COMPUTATION PRECAST TUNNEL
COMPUTED BY JF CHECKED BY D.D. DATE DEC. '76

SUMMARY OF COMPUTER RESULTS

9" LINER W/ 9" LOADS

SECT.	L.C.	MAX. (+) MOM.		MAX (-) MOM.		MAX. THRUST		MAX. SHEAR	
		M (FT. K)	T (K)	M (K·FT)	T (K)	M (K·FT)	T (K)	V (K)	T (K)
1	1	0.25	68.18	0.46	67.72	0.04	70.42	0.31	67.78
	2	2.36	7.95	1.73	5.37	-0.03	12.41	0.74	8.43
	3	2.81	73.98	1.83	70.85	-0.07	78.04	1.34	74.35
	4	5.85	13.11	11.94	11.62	+7.19	17.78	4.53	12.31
	4'	0.70	25.58	0.45	25.56	+0.80	25.59	0.47	25.56
→	5'	1.05	138.52	1.11	138.81	-0.03	139.31	0.79	137.98
2	1	← SAME AS SECT. 1 →							
	2	5.61	19.53	4.33	13.28	0.08	27.82	1.74	20.63
	3	6.66	86.02	4.63	78.52	0.12	94.37	2.68	86.88
	4	← SAME AS SECT. 1 →							
	5'	6.08	144.81	4.06	137.35	0.13	151.91	2.87	145.68
3	1	← SAME AS SECT. 1 →							
	2	13.36	46.61	10.35	31.78	0.20	63.84	4.06	49.19
	3	15.02	111.78	10.86	96.74	0.24	129.43	4.93	116.36
	4	~~14.55~~ ← SAME AS SECTION 1 →							
	5'	14.55	170.96	10.32	155.48	0.25	187.88	5.34	175.12
	+M -	Tension on outside (Rock) Face of Liner.							
	-M -	Tension on inside (Open) Face of Liner.							

SUBJECT PARK RIVER
COMPUTATION PRECAST TUNNEL –
COMPUTED BY JF CHECKED BY D.D. DATE 12-6-76

AVERAGE

COMPR. STRESS

9" LINER W/ 9" LOADS

SECT.	LC	f'c	T_{MAX}	$f_{c\,ave}$	$f_{c\,all}$
1	1	5000	70.42	652.0	1250
	2		12.41	114.9	1250
	3		78.04	722.6	1250
	4		17.78	164.6	1250
	4'		25.59	236.9	1250
	5'		139.31	1289.9	1667
2	1	5000			1250
	2		27.82	257.6	1250
	3		94.37	873.8	1250
	4				1250
	5		151.91	1406.6	1667
3	1	5000			1250
	2		63.84	591.1	1250
	3		129.43	1198.4	1250
	4				1250
	5		187.88	1739.6	1667 ← A MERE 4% OVERSTRESS OK

$$f_{c\,ave} = \frac{T_{MAX}}{bt}$$

$b = 12"$
$t = 9"$
$bt = 108\ in^2$

SUBJECT PARK RIVER
COMPUTATION PRECAST TUNNEL
COMPUTED BY JF CHECKED BY D.D. DATE 12-6-76

PRINCIPLE STRESSES

SECT.	L.C.	T (K)	V (K)	τ (PSI)	Fc/2 (PSI)	$\sqrt{(Fc/2)^2+\tau^2}$ (PSI)	Fcomp (PSI)	Ften. (PSI)
1	1	67.78	0.31	4.7	313.8	313.8	627.6	0.0
	2	8.43	0.74	11.2	39.0	40.6	79.6	1.6
	3	74.35	1.34	20.3	344.2	344.8	689.0	0.6
	4	12.31	4.53	68.6	57.0	89.2	146.2	32.2
	4'	25.56	0.47	7.1	118.3	118.5	236.8	0.2
	5	137.98	0.79	12.0	638.8	638.9	1277.7	0.1
2	1							
	2	20.63	1.74	26.4	95.5	99.1	194.6	3.6
	3	86.88	2.68	40.6	402.2	404.2	806.4	2.0
	4							
	5	145.68	2.87	43.5	674.4	675.8	1350.2	1.4
3	1							8.2
	2	49.19	4.06	61.5	227.7	235.9	463.6	~~5.2~~
	3	146.36	4.93	74.5	538.7	543.8	1082.5	5.1
	4							
	5	175.12	5.34	80.9	810.7	814.7	1625.4	~~4.7~~ 4.0

SUBJECT PARK RIVER

COMPUTATION Precast Tunnel

COMPUTED BY JF CHECKED BY D.D. DATE Nov. '76

TRANSFORMED AREA DESIGN CASES

CASE 1

N outside section, $e_o > 0$

CASE 2

N inside SECTION but outside Kern $e_o < 0$

CASE 3

N inside Kern, Min. REINF.

CORRELTION FACTOR (INSIDE FACE ONLY)

FORMULAS FOR STRESS AND STRAIN by Ray Roark pg. 164 TABLE VII CASE 1

$$F = k_i = 1 + 0.5 \frac{I}{b c^2} \left[\frac{1}{R-c} + \frac{1}{R} \right]$$

$$F = 1 + \frac{0.5 \times 0.083}{1.0 \times 0.375^2} \left[\frac{1}{11.375 - 0.375} + \frac{1}{11.375} \right]$$

$F = 1.02$ (apply only to inside face)

$\boxed{\text{SOLVE FOR S by trial and error}}$

$$\tfrac{1}{2} b f_c S(S/3 + e_o) + \left(\frac{S-d'}{S}\right)(2n-1) f_c A_s' (d' + e_o) - \left(\frac{d-S}{S}\right) n f_c A_s (d + e_o) = 0$$

$$f_c = \frac{S N F^{*}}{\frac{bS^2}{2} + (S-d')(2n-1)A_s' + (d-S)\, n A_s}$$ CASE 1 & 2

$$f_s = \left(\frac{d-S}{S}\right) n f_c F^{**}$$ (TENSION STEEL) CASE 1 & 2

$$f_s' = \frac{(S-d')}{S} 2n f_c F^{*}$$ (COMP. STEEL) CASE 1 & 2

$$f_c = \frac{N}{bD} + \frac{6M}{bD^2}$$ CASE 3

Use MIN. REINF. FOR CASE 3

* APPLY WHEN M is POS.

* APPLY WHEN M is NEG.

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
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SUBJECT PARK RIVER

COMPUTATION PRECAST TUNNEL - SUMMARY OF STRESSES

COMPUTED BY JF CHECKED BY D.D. DATE 12-7-76

PAGE F59

— TUNNEL DESIGN —

SECT.	LC	CASE	M (FT. K)	T (K)	T/A + M/S (PSI)	T/A − M/S (PSI)	e (in)	e_o (in)	DES. CASE	S (in)	fc (PSI)	fs (KSI)	fs' (KSI)	As (in²)	As' (in²)
1	1'	+M	0.25	68.18	649.8	612.8	0.04		3	—	662.8	—	—		
		−M	0.46	67.72	661.1	593.0	0.08		3	—	661.1	—	—		
		T	0.04	70.42	655.0	649.1	0.01		3	—	668.1	—	—		
	2	+M	2.36	7.95	248.4	−101.2	3.56	−0.94	2	4.1	346.9	2.0	0.7	0.31	0.31
		−M	1.73	5.37	177.9	−78.4	3.87	−0.63	2	3.0	294.9	5.2	2.1	0.31	0.31
		T	0.03	12.41	117.1	112.7	0.03		3	—	119.4	—	—		
	3	+M	2.81	73.98	893.1	476.8	0.46		3	—	911.0	—	—		
		−M	1.83	70.85	791.6	520.5	0.31		3	—	791.6	—	—		
		T	0.07	78.04	727.8	717.4	0.01		3	—	742.4	—	—		
	4'	+M	0.70	25.58	288.7	185.0	0.33		3	—	294.5	—	—		
		−M	0.45	25.56	270.0	203.3	0.21		3	—	255.6	—	—		
		T	0.80	25.59	296.2	177.7	0.38		3	—	302.1	—	—		
	5'	+M	1.05	138.5	1360.2	1204.6	0.09		3	—	1387.4	—	—		
		−M	1.11	138.8	1367.4	12030	0.10		3	—	1367.4	—	—		
		T	0.03	139.31	1292.1	1287.7	0.00		3	—	1317.9	—	—		
2	2	+M	5.61	19.53	596.4	−234.7	3.45	−1.05	2	4.2	824.8	4.5	2.4	0.31	0.31
		−M	4.33	13.28	443.7	−197.8	3.91	−0.59	2	3.0	729.2	4.4	5.2	0.31	0.31
		T	0.08	27.82	263.5	251.7	0.03		3	—	266.8	—	—		
	3	+M	6.66	86.02	1289.8	303.2	0.93		3	—	1315.6	—	—		
		−M	4.63	78.52	1070.0	384.1	0.71		3	—	1070.0	—	—		
		T	0.12	94.37	882.7	864.9	0.02		3	—	900.4	—	—		
	5'	+M	6.08	144.81	1791.2	890.5	0.50		3	—	1827.0	—	—		
		−M	4.06	137.35	1572.5	971.0	0.35		3	—	1572.5	—	—		
		T	0.13	151.91	1416.2	1396.9	0.01		3	—	1444.5	—	—		
3	2	+M	13.36	46.61	1421.2	−558.1	3.44	−1.06	2	4.3	1907.5	9.9	5.0	0.31	0.31
		−M	10.35	31.78	1060.9	−472.4	3.91	−0.59	2	3.0	1745.0	10.3	12.6	0.31	0.31
		T	0.20	63.84	605.9	576.3	0.04		3	—	618.0	—	—		
	3	+M	15.02	111.78	2147.6	−77.6	1.61	−2.89	2	8.5	2087.9	−1.7	17.4	0.31	0.31
		−M	16.86	96.74	1700.2	91.3	1.35		3	—	1700.2	—	—		
		T	0.24	129.43	1216.2	1180.6	0.02		3	—	1240.5	—	—		
	5'	+M	14.55	170.5	2656.5	500.9	1.02		3	—	2709.6	—	—		
		−M	10.32	155.5	2204.3	675.4	0.80		3	—	2204.3	—	—		
		T	0.25	187.88	1758.2	1721.1	0.02		3	—	1793.4	—	—		